GEOLOGICAL AND GEOTECHNICAL CONDITIONS OF NAIROBI SUBSURFACE

BY

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DECLARATION

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This thesis is my original work and has not been presented for a degree in any other university.

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ABSTRACT

The City of Nairobi is the capital as well as the social, economic and communication hub of Kenya. It is located on the eastern flank of the East African Rift Valley. The study area is mainly underlain by pyroclastic volcanic rocks intercalated with sediments that were deposited in a large lake or series of lakes that extended from Ol Doinyo Sabuk to almost Kajiado. Owing to the geologic origin, the Nairobi subsurface is not homogeneous. Constructors have reported variable subsoil at building sites, defects in structures that can be related to foundation conditions or adjacent excavation and groundwater flow into excavations and basements. There is also fear that the extensive groundwater extraction could result in depletion of aquifers and eventually cause surface subsidence. The main objective of this study was to investigate the geological and geotechnical characteristics of the subsurface materials and recommend areas where shoring and underpinning works are required to safeguard against effects of ground movement, sensitive and variable subsoil and groundwater drawdown.

Primary data was collected through standard methods of data collection. This included observation and sample collection from construction sites, investigation of causes of distress in structures, core logging of geotechnical and water supply boreholes drilled during the period of study and geophysical resistivity survey. Geophysical resistivity surveys were carried out to relate geoelectric properties to mechanical properties using various formulae obtained from experimental studies elsewhere. The electrical sounding was in most cases carried out adjacent to boreholes to compare the subsurface profile with the geophysical properties. Secondary data was obtained from archival records and reports, structural designs, publications and conference papers as well as from people who previously carried out work in the study area. The secondary data includes data on geology, rainfall, hydrogeology, earthquakes, structural foundation design, shoring and underpinning operations and geotechnical and failure investigations.

In the analysis, the geological map, rivers and contours were digitized for use in GIS; spatial and temporal rest level variations were analysed using Surfer and GIS software packages. Subsurface profiles for various localities were plotted using Strater software and probable settlements were calculated. Fractures were traced using borehole logs and Didger software. The following geotechnical properties were analysed: grading; consistency; expansion and collapse; direct shear and triaxial shear; consolidation; bearing capacity and allowable pressures.

The study concluded the following: groundwater rest levels have dropped with an average of 73 m in the last 80 years and estimated settlement of 0.034 m to 5.9 m could result from groundwater depletion from aquifers and clay aquitards over a long period of time; there seems to have been many centres of volcanic activity during formation of Kerichwa Valley Series leading to formation of materials with variable geotechnical properties; variable subsoil at construction sites and in adjacent boreholes can be related to fractures that are concealed by swamp soils and deep alluvium; the geologic materials have been insufficiently characterised by geotechnical data but the gaps in knowledge were bridged by analysis of logs recovered from water supply wells; the engineering properties of the soils vary widely and are related to the parent material; the difference in bearing capacities of weathered and fresh rock ranges between 10 and 30 times; the allowable pressures on the subsoil range between 0 kN/m^2 to 26537 kN/m^2 while the design pressures range between 0 kN/m^2 and 6000 kN/m^2 ; Weathered/decomposed tuffs, agglomeratic tuffs, silty clays and red clays are liable to collapse in excavations; Distress is common in structures supported on concealed fractures, moisture sensitive soils, thin/ variable laterites, fill and inclined profiles.

The study recommends the following: Drilling of monitoring boreholes in the vicinity of meteorological stations to estimate natural recharge and determine safe yield for the purpose of controlling rest level falls; installation of a few real-time settlement meters to monitor sites with high yielding wells and areas with potential of large settlements; use of two-dimensional tomography to further investigate the presence of concealed fractures and for routine site investigations; shoring and/or underpinning operations when making vertical cuts adjacent to existing shallow foundations to control movements; and, use of the successful construction and shoring/underpinning methods discussed here to mitigate distress in structures. It also recommends serious attention to the conditions of the subsoil before structures are constructed to avoid potential of hazards and disasters in the construction industry like what has been experienced in Nairobi in the last few years.

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DEDICATION

To my late brother Evans, who always believed in me and sacrificed so much for us, my late mother Perpetuah who overwhelmed me with her love and her persistent question "when are you completing the program?" and my little brother Evan for his friendship and usual question "what are you reading?" I wish you all lived to see me graduate.

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LIST OF ACRONYMS

AIT – Asian Institute of Technology

ASCE- American Society of Civil Engineers

BGL- Below Ground Level

BH – Borehole

BS- British Standards

CBS- Central Bureau of Statistics

CSG - Continuously shaft grouted

CPU - Central Processing Unit

DTM- Digital Terrain Model

GIS- Geographic Information System

ICRAF- International Centre for Research in Agroforestry

KVS- Kerichwa Valley Series

LAS- Lower Athi Series

MAS- Middle Athi Series

MMUST- Masinde Muliro University of Science and Technology

MPa- mega pascals

MR&PW- Ministry of Roads and Public Works

NSSF- National Social Security Fund

NWSSC- Nairobi Water and Sewerage Services Company

OLS- Old Land Surface

RQD- Rock Quality Designation

TCR - Total Core Recovery

UAR- Annual Utilization Rate

UAS- Upper Athi Series

USCS- Unified Soil Classification System

VES- Vertical Electrical Sounding

WHO- World Health Organisation

WRMA- Water resources Management Authority

DEFINITION OF GEOLOGICAL TERMS

- **Extrusive igneous rocks** are formed at the earth's crust as a result of the partial melting of rocks within the mantle and crust. Extrusive Igneous rocks cool and solidify quicker than intrusive igneous rocks. Since the rocks cool very quickly they are fine grained.
- Eruptions of volcanoes into air are termed **subaerial** and when deposited in water they are termed **subaqueous.**
- **Pyroclastic rocks** or **pyroclasts** are clastic rocks composed solely or primarily of volcanic materials. Pyroclastic rocks may be composed of a large range of clastic sizes; from the largest **agglomerates**, to very fine **ashes** and **tuffs**.
- Pyroclastic deposits that are compacted and cemented together are referred to as **tuff.** Welded tuff is a pyroclastic rock, of any origin, that was sufficiently hot at the time of deposition to weld together.
- **Agglomerates** are coarse accumulations of large blocks of volcanic material that contain gravels, cobbles and boulders. Agglomerates are typically found near volcanic vents and within volcanic conduits, where they may be associated with pyroclastic or intrusive volcanic breccias.
- **Lava** is molten rock material that reaches the earth's surface. Lavas also may contain many other components, sometimes including solid crystals of various minerals, fragments of exotic rocks known as **xenoliths** and fragments of previously solidified lava.
- **Phonolite** is any member of a group of extrusive igneous rocks that are rich in nepheline and potash feldspar. The typical phonolite is a fine-grained, compact igneous rock that splits into thin, tough plates which make a ringing sound when struck by a hammer, hence the rock's name.
- **Trachyte** is a light-coloured, very fine-grained igneous rock composed chiefly of alkali feldspar with only minor mafic minerals (biotite, hornblende, or pyroxene). Occasionally minerals of the feldspathoid group, such as nepheline, sodalite and leucite, occur, and rocks of this kind are known as **phonolitic trachytes.**

CHAPTER ONE

INTRODUCTION

1.1 General introduction

The City of Nairobi is the capital as well as the social, economic and communication hub of Kenya. The city is located on the eastern flank of the East African Rift Valley. It is mainly underlain by pyroclastic volcanic rocks intercalated with sediments that were deposited in a large lake or series of lakes that extended from Ol Doinyo Sabuk to almost Kajiado (Matheson, 1966). Owing to the geologic origin, the subsurface in Nairobi is not homogeneous. Constructors have reported variable subsoil at building sites, defects in structures that can be related to foundation conditions or adjacent excavation and groundwater flow into excavations and basements. There is also fear that the extensive groundwater extraction could result in depletion of aquifers and eventually cause surface subsidence.

In general, natural geological materials are difficult to quantify for engineering use because of the various factors that affect their behaviour. For example, soil behaviour is affected by the initial state of stress, direction of loading, drainage conditions and loading rate. The engineering behaviour of a rock depends on its type, the number and location of discontinuities, fractures, joints, fissures, cracks and planes of weakness. Because of variation in the above factors, samples from the same material can exhibit varying engineering properties. Therefore, the important component of characterizing geological materials for engineering purposes involves establishing reasonable property variations and providing properties that represent the best estimates. The qualitative and quantitative description of conditions on or beneath the surface that leads to appropriate facility design or the feasibility of remedial actions is referred to as subsurface characterization.

1.2 Characteristics of the study area

1.2.1 Location of the study area

Nairobi City is located in the central part of Kenya, 500 km from the port of Mombasa where the country borders the Indian Ocean to the east and 400 km from Busia at the Kenya-Uganda Border to the west (Figure 1.1). Currently, the city covers an area of 692 km^2 (Central Bureau of

Statistics, 2009). Nairobi City and the area around it herein referred to as the study area is bounded by longitudes 36°40′00″E and 36°55′00″E and latitudes 01°10′30″ S and 01°25′00″ S.

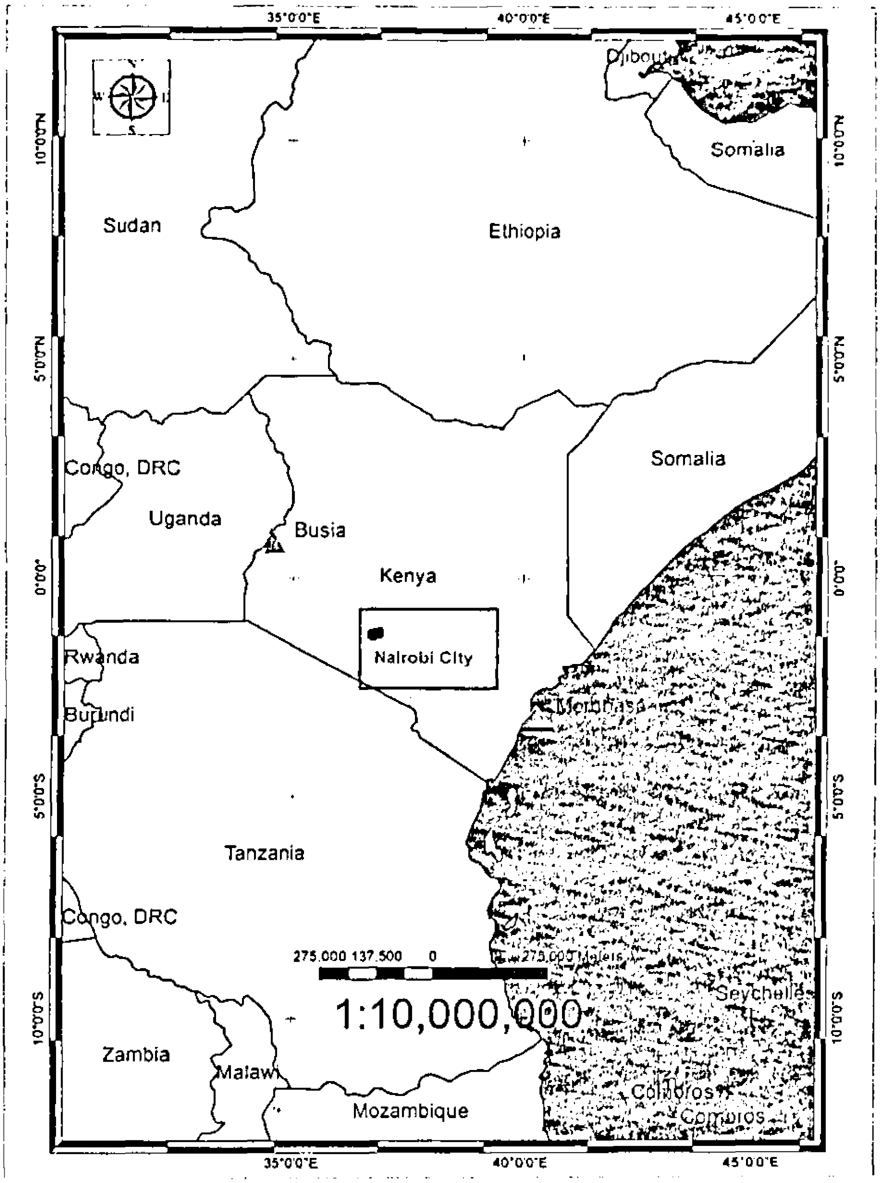

Figure 1.1: Location map of Nairobi City (Atlas.com)

Development of the City of Nairobi started in 1899 during the construction of the Kenya-Uganda Railway Line. The first settlement was started by the Railway Survey Team that pitched tents beside the crystal clear water from the Central Highlands after working through dry arid areas. It was decided that Nairobi be made a storage depot for the railway construction materials and building of permanent structures began. With the depot acting as the nucleus, the population started to grow and by 1905, Nairobi became the capital of Kenya with a population of about 10 000 people. It became a municipality in 1919 and a city in 1950 (Mwangi, 2005). The Nairobi City boundaries have since been revised at least four times.

1.2.2 Population

According to the Republic of Kenya Population and Housing Census, the urban population of Nairobi City reached 3,138,295 million on August 2009 (Table 1.1). This figure does not include the population in the surrounding suburban and rural areas that work in the city or the population that comes to the city to carry out administrative tasks or conduct business and remains in the city for a short period. The estimated growth rate of the city population is about 5.5% per annum; approximately 100 000 new people being added to the city every year. The rate of migration from rural to urban areas has significantly increased due to economic opportunities in urban areas and to the decrease in productive farmlands in rural areas as a result of population growth and climate change.

Table 1.1: Nairobi City population between 1906 and 2009

Year	Area (Ha)	Population	Average increase per annum
1906	1,813	11,512	0
1928	2,537	29,864	834
1931	2,537	47,919	6,018
1936	2,537	49,600	336
1944	2,537	108,900	7,037
1948	8,315	118,976	2,519
1960	68,945	266,795	12,318
1969	68,945	509,286	26,943
1979	68,945	827,775	31,848
1989	68,945	1,324,570	49,679
1999	68,945	2,143,254	81,868
2009	68,945	3,138,295	99,504

(Source: Republic of Kenya, Central Bureau of Statistics, CBS)

1.2.3 Climate

Nairobi has an average altitude of 1695 m above sea level with tropical highland climate. There are two rainy seasons: late March to early June and late September to early December and two dry seasons: late December to early March and late June to early September. The altitude makes for some chilly evenings, especially in the June/July season when the temperature can drop to 10 °C. As Nairobi is situated close to the Equator, the differences between the seasons are minimal though there are years when the dry seasons extend over longer periods.

1.2.4 Structural development

In the face of rapid population growth, it has become increasingly difficult to meet the demand for office space and housing. This has led to development of many medium and high-rise buildings within the city boundaries and in peri-urban areas that are currently being absorbed into urban Nairobi. Some of the structures are being built on marginal land whose capacity to support the substantial weight of structures has not been ascertained. Such areas include former swamps, river banks, former dumpsites and quarries. Developers are also increasing the number of floors in old low-rise structures or altogether replacing them with taller structures.

1.3 Geological setting

The City of Nairobi is located on the eastern flank of the East African Rift Valley. Geologically, the subsurface comprises of volcanic formations cooled under aerial and subaqueous conditions. The volcanic rocks overlie Mozambique Belt metamorphic rocks of Neoproterozoic Era. Figure 1.2 is a geological map showing the full extent of the study area while Figure 1.3 is the key to the geological succession.

The Mozambique Belt rocks consist of ancient sediments that were metamorphosed as a result of high temperatures and pressures in the late Precambrian to Lower Palaeozoic times. Following the metamorphism and folding of the rocks, the area was subjected to erosion lasting for more than 400 million years, leaving an erosion surface dated to end Cretaceous Age (Ministry of Works, 1969).

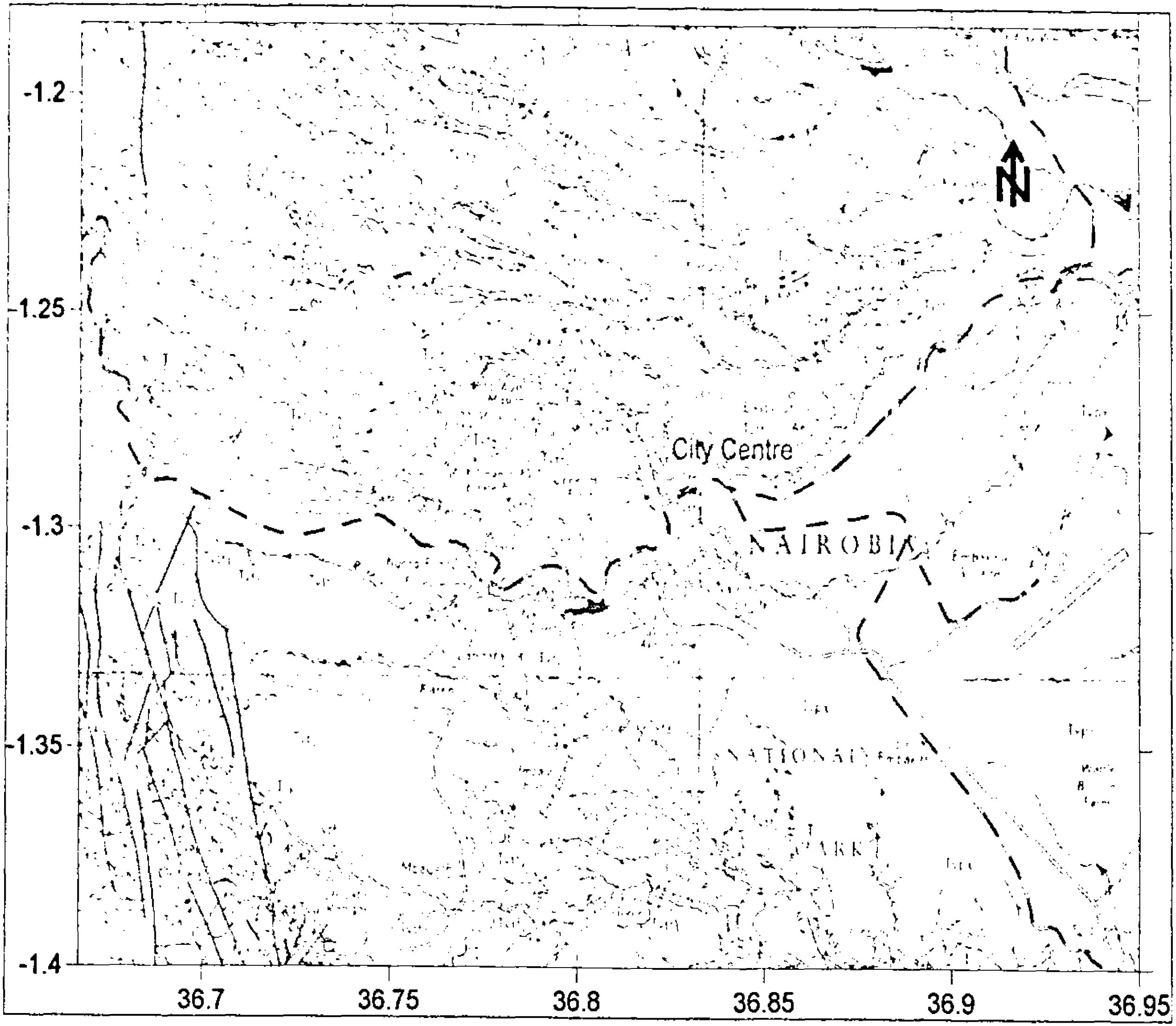

Figure 1.2: Geological map of Nairobi City (After Saggerson, 1991)

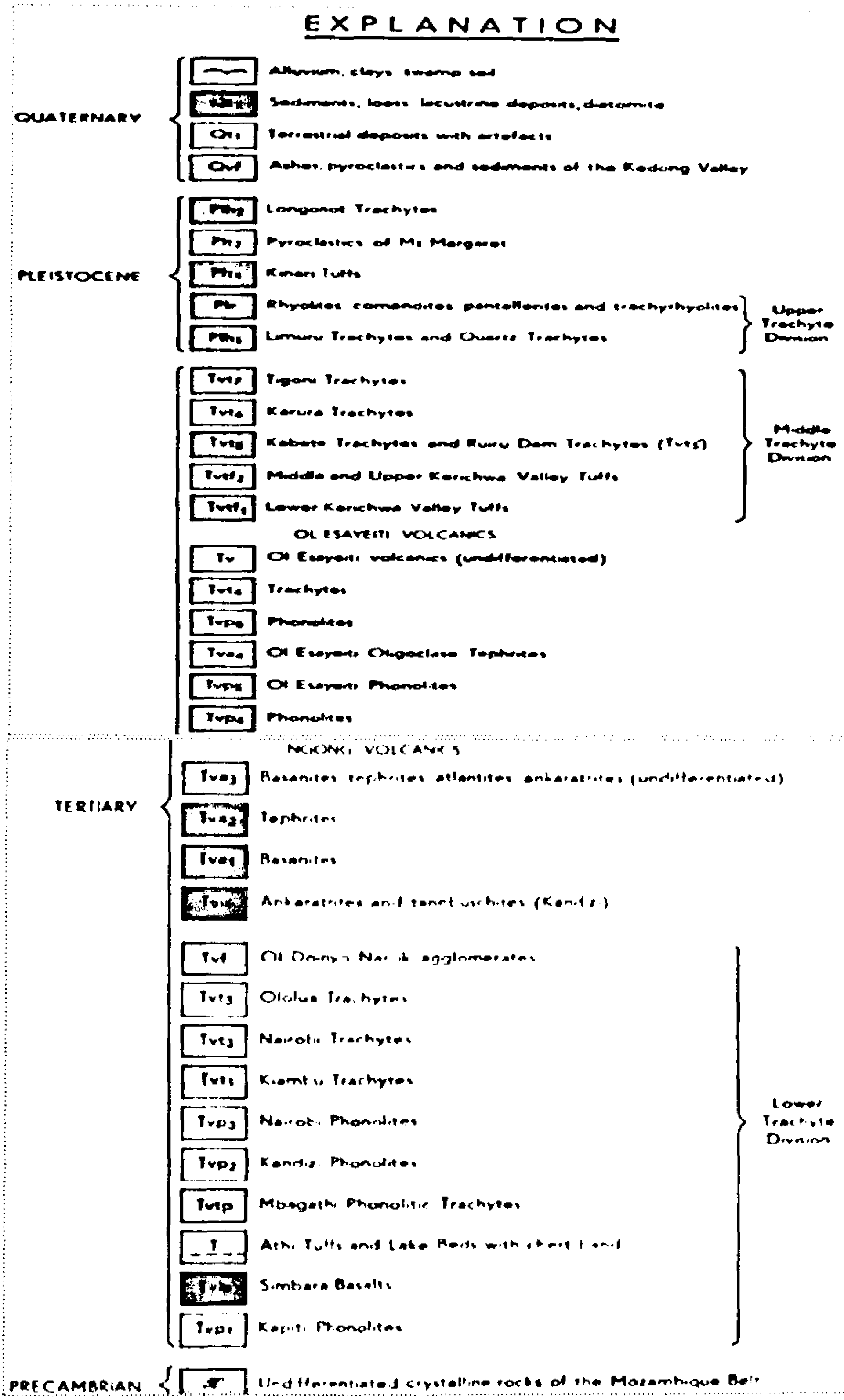

Figure 1.3: Key to the geological map of Nairobi City (After Saggerson, 1991)

In the Upper Miocene times, phonolitic lava flowed across the eroded metamorphic surface from the edge of the newly formed Rift Valley. This lava is known as Kapiti Phonolite since it underlies the Kapiti Plains. The Kapiti phonolite is a rock with large white crystals of feldspar and waxy-looking nephelines set in a fine-grained dark-green to black ground mass (Saggerson, 1991). Some exposures reveal the vesicular nature of parts of the phonolite and small patches of calcite and zeolite-filled amygdales are common.

The Kapiti Phonolite is overlain by pyroclastic rocks with interbedded lacustrine sediments. The succession is referred to as Athi Series Tuffs and Lake Beds or simply as Athi Series because the largest exposures are in Athi River area. The colour of the tuffs ranges from black to grey to yellow but are generally fine grained. The lake beds contain plant remains and show signs of desiccation. The Athi Series has variable thickness depending on the shape of the erosion surface at the time of deposition with a maximum thickness of 305 m at Nairobi City Centre (Gevaerts, 1964).

Generally, the Athi Series is characterized by the presence of obsidian but with much lateral variation. The Series is divided into three parts namely: the Upper Athi Series, Middle Athi Series and Lower Athi Series. The Upper Athi Series (UAS) mainly consists of sandy sediments and tuffs, clays being subordinate. The Upper Athi Series also includes few intercalated black non-porphyritic basalt flows. The UAS is generally soft and friable in character. A hard yellow tuff band (6 m) forms a good marker horizon.

In between the UAS is Mbagathi Phonolitic Trachyte, which is porphyritic lava with tabular insets of feldspar. The most striking feature of the Mbagathi Phonolitic Trachyte is its texture of crowded feldspar laths set in a grey-brown matrix, the colour of which is emphasized by rusty brown weathering and alteration products. In many places, Mbagathi Phonolitic Trachyte is vesicular and the feldspars, which are up to a centimetre in length, are flow oriented (Saggerson, 1991). Mbagathi Phonolitic Trachyte can be traced in boreholes at Kabete, Karen, Ngong and Ongata Rongai. The most easterly extent is at Nairobi National Park.

Middle Athi Series (MAS) consists of basalt lava flows, sands and agglomerates. The lavas show abundant insets of feldspar and the sands often have a clay matrix (Gevaerts, 1964). Lower

Athi Series (LAS) are predominantly clayey deposits between the basaltic Middle Athi Series and the uppermost flow of the Kapiti Phonolite (Saggerson, 1991). In some places the LAS lies directly on the metamorphic rocks.

Overlying the Athi Series rocks are the Nairobi and Kandizi phonolites. Nairobi Phonolite is dark- grey, porphyritic lava with tabular insets of feldspar and a few flakes of biotite. The lava is distinguished in drilling samples by the presence of biotite. The Kandizi Phonolite is non-porphyritic lava with only very sporadic insets of feldspar, biotite being absent. The groundmass of Kandizi Phonolite is like that of the Nairobi Phonolite.

In some places, Nairobi Phonolite is overlain by Nairobi Trachyte. Nairobi Trachyte is greenish grey lava that is occasionally porphyritic with tabular phenocrysts of feldspar. The groundmass is fine-grained with a silver lustre. Nairobi trachytes are traced in boreholes in Karen, Langata, Upper Hill, Kilimani Lavington, Riruta, Kasarani and Kiambu. Several flows of the Nairobi Trachyte have been encountered in boreholes at Upper Hill with a total thickness of 91 m. A borehole at Kenyatta National Hospital penetrated five flows, each with a vesicular upper contact. Three flows separated by sediments or tuffs were recognized at Mbagathi (Gevaerts, 1964). At Upper Hill, Nairobi Trachyte is separated from Nairobi Phonolite by a few metres of agglomeratic tuff. Nairobi trachytes have great affinities to phonolitic rocks though lacking in nepheline.

Ngong' Volcanics overlie Kandizi Phonolite and overlap onto the Mbagathi Phonolitic Trachyte. They are dark-grey lavas of basalt and nepheline interbedded with sands. In boreholes they occur immediately below Nairobi Trachyte. Kerichwa Valley Series (KVS) overlies the Nairobi Trachyte and nearly every other older formation. The series comprises of pumice-rich trachytic tuffs and agglomerates that have some resemblance to some of the Athi Tuffs and Lake Beds. They are found in boreholes at Wilson Airport, Ongata Rongai, Bulbul, Dagoretti, Ruaraka, Eastleigh, Karura, Muthaiga and Mathare.

The KVS tuffs range from cemented fine-grained, wind-sorted pumiceous ash to agglomeratic tuffs with rock fragments up to 0.5 m size. The tuffs are referred to as "agglomeratic tuffs" not agglomerates because of the larger proportion of fine material that forms the rock. The colour of

Kerichwa Valley tuffs is generally yellow, grey or black. The deposits of KVS buried a pre-existing landscape, the former valleys of which are now being re-excavated to reveal Nairobi Trachyte and Nairobi Phonolite. The KVS is up to 20 m thick at the City Centre where it overlies Nairobi Phonolite.

The Series cannot be traced to any volcanic vent and it is probable that they are the result of rapid pressure release following rift faulting and graben-forming collapse. It is thought that one of the centres of volcanic activity during this period was located somewhere in the City Centre because it is unlikely that the large agglomerate fragments were ejected from a far away vent (Ministry of Works, 1969). At one time during the period of this volcanic activity, there seem to have been two main drainage systems through Nairobi City Centre (Gevaerts, 1964). One is on a line running in the general direction of Moi Avenue, which possibly was the old line of Nairobi River. The other one is in a direction from parliament towards the Railway Station. These valleys were subsequently filled with tuffs and Nairobi River changed its position to the present course. The presence of conglomerates and fluvial deposits in the series suggests more than one flow.

The north of the study area is covered by Kabete Trachyte, Karura Trachyte and Limuru Quartz Trachyte. These trachytes overlie the Kerichwa Valley Series that is found to thicken to the north and north-west with an increase in obsidian content. Kabete Trachyte is greenish-grey porphyritic rock that weathers to soft grey colour and has similarities with Nairobi Phonolite. It has limited lateral extent and has a maximum thickness of 30 m in Kabete. Karura Trachyte is fine-grained, dull grey to lustrous rock, similar to Nairobi Trachyte but higher in succession and spotted when weathered.

1.4 Statement of the problem

Geotechnical and construction records indicate that highly variable volcanic materials and sediments are encountered at construction sites in Nairobi City (Ministry of Works, 1987). Development of buildings in such high variability soils requires geotechnical studies to obtain accurate information for foundation design (Terzaghi *et al.*, 1996). However, most high-rise residential buildings and medium-rise commercial enterprises in Nairobi City are designed using information obtained from trial pits that are done to firm strata.

Development of several high-rise buildings on alternating layers of weak and strong subsoil can cause "group failure". Group failure may occur when a group of individually safe foundations rest on a relatively strong layer overlying a thick layer of weak material. The foundations soils settle due to the weight of the individual buildings, termed "inherent settlement" and can be accompanied by the additional settlement caused by loads on neighbouring foundations termed "interference settlement" (Comrie, 1961). The new high-rise buildings in the city may be individually safe but when their bearing pressures are combined with those of adjacent structures, they affect deeper subsoil and can be liable to group failure.

Groundwater in the pores assists the subsoil in supporting structural loads and thus reduces the effect of foundation pressures on the subsoil. Extensive groundwater abstraction is taking place across Nairobi City causing drawdown (Foster & Tuinhof, 2005). Whenever the water table is lowered, the soil particles adjust as they occupy the empty pore spaces and thus decrease in volume; the resulting displacements produce settlement of the ground surface that is roughly proportional to the descent of water (Sun, 1997). When the ground settles, it causes distress or damage to the structures that it supports.

There is increasing difficulty in getting parking space within the Nairobi City Centre and this is causing many tenants to seek alternative office space with ample parking. Faced with critical shortage of tenants, new property developers within the City Centre will consider constructing multilevel parking basements. But from drillhole information, weak soils have been encountered at some shallow depths leading to backfilling of boreholes up to 20 m (Gevaerts, 1964). Excavations with vertical cuts in such soils would lead to construction difficulties and necessitate the need for shoring and underpinning operations. A study of subsurface strata done down to depths penetrated by water supply wells has bridged the gap in knowledge about subsurface conditions beyond depths confirmed in excavations and geotechnical site investigation boreholes.

Excavation for foundation placement adjacent to an existing structure removes the side support to the foundation of the latter. Removal of foundation support causes deformation that commonly takes the form of a subsidence of the area surrounding the excavation; an inward movement of the soil at the sides and an upward movement of the soil beneath the bottom (Coduto, 2001).

Structures supported by foundations resting on the material that deforms experience corresponding movement; they settle and move towards the excavation (Tomlinson, 1995). This movement beneath a building is manifested in form of bulging floors, cracked walls, and jammed doors and windows. Some structures in Nairobi City have experienced distress that is attributable to adjacent excavation.

Foundations for most old buildings in the city are supported on Kerichwa Valley Series deposits, some with bearing capacity that cannot sustain extra load (Gevaerts, 1964). In spite of this, some property developers increase the number of storeys in response to increasing demand. Additional loads on the structures lead to higher foundation pressures and can induce new settlement and distress in structures. This study analysed the engineering properties of the various subsurface materials encountered at building sites so that effects of increasing loads on old buildings can be evaluated and handled accordingly.

1.5 Objectives of the study

Main objective

To investigate the geological and geotechnical characteristics of the subsurface materials and recommend areas where shoring and underpinning works are required to safeguard against effects of ground movement, sensitive and variable subsoil and groundwater drawdown.

Specific objectives

1. To study the variation in groundwater rest levels with time and location and their effects on the stability of civil engineering structures in the city.
2. To analyse the subsurface profile, total thickness of aquifers and estimate the settlement that might result from groundwater extraction.
3. To establish the reasons for sudden changes in subsoil type at construction sites and water supply boreholes within localities.
4. To determine the engineering properties and bearing capacities of the various geologic materials for use in design of structures in the city.
5. To determine the strata liable to collapse at construction sites or settle as a result of physical weight of high-rise buildings so as to determine the most appropriate methods of shoring and underpinning foundations.

6. To identify the causes of defects in structures and to propose methods of preventing them.

1.6 Justification and Significance of the study

The City of Nairobi is progressively becoming highly populated and the development of high-rise residential and commercial structures is replacing old low-rise buildings. The high building density in some parts of the city has resulted to narrow structures that are sensitive to demolition, excavation and construction operations. Furthermore, most of these structures cover the entire footprint of the site and therefore most of their foundations and foundation walls are exposed during adjacent construction. Thorough knowledge of the foundation conditions of the subsurface is necessary for successful shoring or underpinning of such structures. One of the objectives of the study was to analyse the engineering properties of the various strata in the city based on results of laboratory tests, construction and drilling experiences so as to determine the property ranges and pinpoint subsurface materials that require shoring and underpinning works during construction.

Most foundations depend upon the uppermost soil layers to provide sufficient bearing capacity to support the structure and keep the foundation stable. If the bearing soil is weak or non-uniform and is insufficiently recognized prior to construction, the foundation is subject to failure as the supporting soil adjusts to the various pressures. Analysis of the engineering properties of the subsurface provides information useful in the design of structures in high variability areas. Availability of this engineering data can improve the methods used as protection/ mitigation measures prior to the start of construction operations in Nairobi City.

Whereas some research has been done on the effect of groundwater extraction on water rest levels (Gevaerts, 1964; Mailu, 1987; Foster &Tuinhof, 2005), there is inadequate information on the effects of the observed water table lowering on structures. Evaluation of the consolidation-settlement characteristics of the subsurface materials is needed to shed some light on the question of what will be the long term consequences of groundwater level lowering. This study has investigated the current effects of intensive extraction of groundwater and calculated long term settlement that can result from groundwater depletion thus bridged the gap in knowledge.

An unpublished report on settlement problems in Shanghai City in China noted that the rate at which the older city areas were sinking picked up at the beginning of the 1990s when many high-rise buildings were built. The Shanghai report notes that varying heights and the locations of high-rise structures produce uneven pressure on the ground which in turn creates uneven subsidence that may lead to safety risks. According to the Shanghai Geological Research Institute, excessive groundwater pumping contributes to 70 per cent of Shanghai surface subsidence, with the remaining 30 per cent created by the physical weight of skyscrapers. This study has investigated settlement resulting from the weight of skyscrapers in Nairobi City Centre using real-time measurements taken at specific locations in order to postulate the potential effects that the combination of groundwater extraction and weight of skyscrapers might have on the subsoil.

In this study, investigations have been done on a number of sites with deep soft and sensitive soils where structural defects and foundation failures are prevalent. The results describe the construction methods that have been applied to minimize/remedy total and differential settlement and distress in buildings. It is therefore hoped that this study will go a long way in sensitizing our urban developers on the real need for thorough geological and geotechnical investigations at construction sites to avoid future hazards and imminent loss of lives.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Review of literature presented in this chapter is in three parts. The first part discusses the history of groundwater extraction in Nairobi and highlights some cases of ground subsidence due to groundwater withdrawal across the globe. The subsidence/settlement cases are used to express the importance of studying the groundwater rest level variations and the settlement that can result from groundwater extraction in Nairobi City. The part also discusses the effects of groundwater on design and construction of structures in Nairobi City. The second part discusses subsurface characterisation and relates the results of various geotechnical studies that have been carried out elsewhere to the significance of the present study. The third part is a short review on shoring and underpinning of foundations depending on the type of structure and subsurface conditions.

2.2 Nairobi Groundwater

Groundwater is an important element of the overall city water supply through a large number of publicly and privately-operated boreholes. Surface water supply for Nairobi is undertaken by Nairobi Water and Sewerage Services Company (NWSSC). Most of the water is conveyed by gravity from Ndakaini and Sasumua Dams located about 100 km north-west of Nairobi. The bulk water supply is not reliable during periods of drought, and is also endangered by reservoir siltation associated with catchment deforestation. The supply problem is further aggravated by the inadequate distribution system, which results in about 50% losses due to leakage and illegal connections (Foster &Tuinhof, 2005).

Figure 2.1 shows a geological map on which 1600 out of 2500 boreholes in Nairobi area are plotted. The geological units on this map are as explained on Figure 1.3. Most boreholes are concentrated in Karen, Langata, Lower Kabete, Ruaraka, Muthaiga, Westlands and Industrial Area. Due to the frequent water rationing, Water Resources Management Authority (WRMA) receives, on average, thirty requests for drilling boreholes every month. These requests are mainly for parts of the city that have traditionally depended on surface water supply from NWSSC.

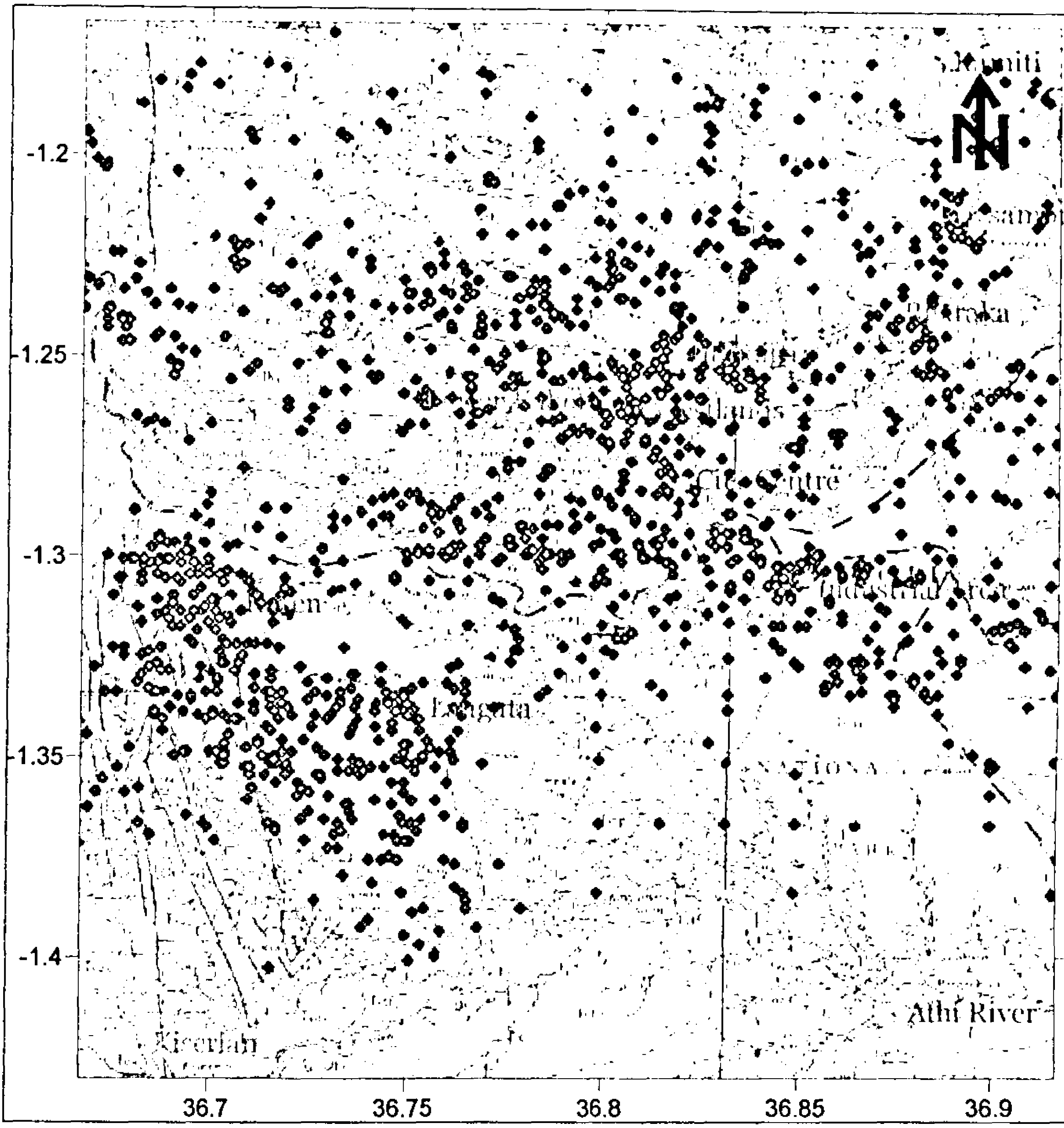

Figure 2.1: Borehole distribution in Nairobi City (map after Saggerson, 1991)

Exploration and drilling for groundwater in Kenya began in December 1927 when a farmer by the name Briscoe drilled two holes at Roysambu (indicated on Figure 2.1) to depths of 20 m and 22 m. By 1934, 190 such boreholes had been drilled (Gevaerts, 1964). The peak of drilling in pre-independence Kenya was in 1950-1951 due to change of the Water Act introducing subsidy for drilling of boreholes. One hundred and sixty nine (169) water supply wells were drilled in two years and intensive abstraction of groundwater took place all over the city. The extraction caused a 13 m fall in the artesian pressure of borehole C-2321 in Kamiti zone within four years after completion. The same kind of drawdown occurred at Kahawa Barracks and at East African Breweries in Ruaraka area where the density of drilling and rate of abstraction were exceptionally high.

In 1953, the government declared Ruaraka area and its immediate surroundings a conservation area, with the object of maintaining a better control of the groundwater resources. A groundwater observation network was established within the conservation area. In 1958, the conservation area boundaries were extended to include the peri-urban areas around Nairobi and part of Kamiti area. Outside the conservation area no restrictions of any kind were placed on boreholes that were drilled more than a kilometre from any existing borehole, but in the conservation area, no borehole would be sunk and no groundwater was to be abstracted without the permission of the Water Apportionment Board.

Before large-scale groundwater development took place in the vicinity Nairobi City, there used to be water flow within the aquifers from the west through the deep, confined aquifer section. The flow emerged in the east near Athi River and boreholes drilled to Kapiti Phonolite (e.g. C-252 and C-498) encountered artesian conditions at the contact with the Athi tuffs and sediments. Artesian pressures were also encountered in Kamiti area in the east. Gevaerts (1964) observed groundwater levels in various boreholes drawing from KVS and Athi Series aquifers in 1963. He noted water level rises in boreholes drawing from KVS aquifers after rainy seasons indicating adequate replenishment while there were no rises in boreholes tapping from Athi Series aquifers. Gevaerts (1964) also performed continuous large scale abstraction followed by 12 hours of monitoring of some boreholes tapping from confined aquifers at Kahawa, Ruaraka and Athi River. He observed no water level recovery during the 12 hours of monitoring. Gevaerts concluded that the depressions were permanent and attributed the falls to depletion of storage and permanent compaction.

Explaining further this phenomenon of permanent compaction in aquifers is a study by Poland *et al.* (1972). They noted that the usable storage capacity of the aquifer system, defined as the volume that can be taken from or recharged to the system, is not changed appreciably by the compaction of the aquitards but the specific storage is greatly reduced for later cycles. As fluids are withdrawn from porous media, pore-fluid pressures decrease. Because deformation of porous media is controlled by effective stress, a decrease in pore-fluid pressure causes a decrease of pore volume. When effective stress exceeds the yield strength of the granular skeleton of the media, the compaction is permanent and irreversible.

In 1972, the World Health Organisation (WHO) carried out a feasibility study on augmenting the Nairobi surface water supply with groundwater. It was found that the major problem of the groundwater was its characteristic high fluoride content (Hove, 1973). The study recommended that groundwater be mixed with surface water in a ratio of 1:1 to make it suitable for domestic use. City Council of Nairobi adopted the findings by WHO and drilled a number of wells to supply certain sectors of the city that were not connected to the main distribution system. By 1985 most of these boreholes were closed down and the corresponding areas shifted to surface water supply (Mwangi, 2005).

A more recent study by Foster & Tuinhof (2005) reports that biannual groundwater-level measurements in a 275 m deep borehole (C-2730) during 1958-1996 by a private company showed a decline that started in 1970 and reached 40 m in 1996. Comparison of data from the Ministry of Water and Irrigation (MW&I) for new water wells drilled at different dates during the period 1950-1998 also indicated a substantial lowering of groundwater level in the upper aquifer units. Groundwater level hydrographs for measurements made in 1970 -1975 also pointed to a similar trend. Foster & Tuinhof (2005) estimated the annual total abstraction in the city at 12 million cubic metres from 1400 boreholes by 1980 and 36 million cubic metres from 2250 boreholes by 2002.

Wells drilled during pre-colonial times encountered water sufficient for domestic supply at depths between 20 and 60 m. New wells drilled adjacent to the old wells have encountered minimal water in the upper aquifers as a result of extensive abstraction. Wells tapping water from depths above 60 m have been deepened while those that have not been deepened are out of operation. There has been a public concern that the amount of groundwater being extracted by the existing 2,500 boreholes in Nairobi Area could lead to depletion of aquifers and cause subsidence/settlement of the ground surface.

A number of cities in the world have been affected by subsidence resulting from groundwater extraction. According to a geological survey by Xu *et al.* (2004), 46 cities in China are sinking due to the excessive pumping of groundwater. Shanghai, for example, started pumping groundwater in 1860, and by the early 1960s, over 200 million cubic metres were being used annually. As a result, the 144 square-metre old district of Shanghai sank an average of 1.75 metres from 1921 to 1965. Shanghai alone has suffered direct economic losses of US$35.1 billion in the last 40 years from destructive tidal waves, floods and other surface subsidence-related disasters. Extraction of groundwater for irrigation purposes has caused subsidence in farmlands in some areas such as Suzhou where the land is more than a metre lower than the surrounding surface water level during rainy seasons. Farmers have to spend a great deal of money to drain off their flooded fields every year.

Japan has the largest number of subsiding areas in the world according to Yamamoto (1977). The number of subsiding areas in Japan had reached 40, and was still increasing. Most of the subsidence was related to groundwater withdrawal from heavily populated topographically low areas. Ten chief subsidence areas in Japan border the ocean. Osaka City where much of the land is reclaimed from the sea is the most affected (Mimura and Young, 2008).

The historic City of Venice, for instance, lies on an island within a lagoon, offshore from the north coast of Italy with the Adriatic Sea to the south. It is underlain by thick sedimentary deposits of up to 1000 m. The top 350 m comprise six sand aquifers interbedded with deposits of silt/clay which act as aquitards. It also has thin layers of peat that are highly compressible (Barnes, 2000). Prior to 1900, artesian heads in Venice City were recorded up to 6 m for wells sunk to shallow aquifers. More intensive water pumping especially after world war II mainly for industrial purposes led to lowering of the piezometric level in the industrial zone up to 20 m while in the city area, it amounted to 10 m. The city experienced subsidence mainly due to exploitation of the aquifer system. An exceptional high tide caused flooding of the city in November 1966.

Bangkok in Thailand is a city that has most of its parts lying about 1 m above sea level. It is built on highly compressible clay of large thickness and excessive groundwater extraction caused consolidation of the clay (Pincharoen, 1977). It has been sinking at the rate of 100 mm/year due to the rapid groundwater drawdown and destruction of mangrove forests (AIT, 1981). It is postulated that if the groundwater extraction continues at the same rate, the city would sink into the Gulf of Thailand in less than 50 years.

In the desert state of Arizona USA, there is evidence of groundwater level decline by between 100 - 200 m, and associated ground subsidence of at least 5 m, causing unequal subsidence and deep land fissures (Newton, 1981). Overdraft in the Santa Clara Valley groundwater basin caused land surface subsidence over an area of about 63,000 hectares with a maximum depression of 3.6 m between 1912 and 1967. Damages resulting from land surface subsidence are estimated to have amounted to over US$ 130 million (Fowler, 1981). In the area of Houston, Texas, groundwater pumping has led to subsidence of about 3 m at the surface, together with a lowering of the groundwater level by about 120 m.

Mexico City in Mexico has also been affected by groundwater drawdown. The city lies within a large basin at an elevation of 2250 m above sea level surrounded by volcanic mountains. Parts of the city are underlain by one of the most difficult and unusual soils in the world, Mexico City Clay (Barnes, 2000). Pumping of the groundwater from the aquifers below Mexico City commenced around 1850, rapidly increased in the 1940s and by 1974 there were 3000 shallow aquifer wells and 200 deep aquifer wells. With this number of wells, abstraction far exceeded recharge and piezometric levels fell up to 30 m in some parts of the city. Settlement records from 1891 to 1973 show that settlements up to 8.7 m occurred in the old city with a maximum rate of 460 mm/yr in 1950. It has been estimated that settlement would potentially reach 20 m (Coduto, 2001).

Much of the settlement in Mexico City has been due to consolidation and secondary compression of the clays down to 50 m. The consequences of the settlements have been disruption of surface infrastructure such as roads, bridges, pavements, services and drainage. In particular, the loss of water supply from dislocated leaking pipes has been considerable. Buildings which were supported on piles taken through the compressible strata have not settled as much, but with the settlement of the surrounding area they appear to be raised out of the ground.

The Leaning Tower of Pisa (Bell Tower) in Italy (perhaps the world's most successful foundation failure) is an example where variable foundation conditions and groundwater affected the stability of the structure. Building of the tower, also known as the campanile, was commenced in 1173 under the leadership of Architect Bannano Pisano. In 1178 when the tower reached a height of four storeys, building was stopped due to construction difficulties and war with Florence. By this time, the tower was leaning north. Nothing further was done for another 100 years. During this 100 year break, the foundation strengthened due to consolidation of the subsoil beneath the weight of the tower. Construction resumed in 1270 under the direction of a new architect Giovanni Di Simone. Although it would have been probably best to tear down the completed portion and start from scratch with a new and larger foundation, Di Simone chose to continue working on the uncompleted tower attempting to compensate for the tilt by tapering the successive storeys and adding extra weight to the higher side. Work ceased again in 1284 due to war with Genoa. Construction continued in 1370 leading to completion. Altogether, the project had taken nearly 200 years to complete.

Both the north and south sides of the Bell Tower continued to settle such that by 1817, the tower had settled about 2.5 m into the ground and tilt was 4.9°. As a result the elegant carvings at the bottom of the tower were no longer visible. To "rectify" this problem, in 1838 a circular trench was excavated around the perimeter of the tower to expose the bottom of the columns. Unfortunately construction of the trench disturbed the groundwater table and removed some lateral support from the side of the tower. As a result, the tower suddenly lurched and added about half a metre to the tilt at the top. In the 1930s, the fascist dictator Benito Mussolini decided the leaning tower presented an inappropriate image of the country, and ordered a fix. His workers drilled holes through the floor of the tower and pumped 200 tons of concrete into the underlying soil, but this only aggravated the problem and the tower gained an additional 0.1 degree of tilt (Barnes, 2000). The nature of the subsoil was completely ignored as no detailed measurements or analyses were carried out for understanding the causes of movements of the structure.

2.3 Subsurface characterisation

Civil engineering structures are supported on natural geologic materials with diverse geotechnical characteristics. Loads from the structures are transmitted to the underlying materials through their foundations. Foundation is that part of a structure that provides support to the structure and the loads coming from it (Tomlinson, 1995). Foundation also includes the soil or rock that ultimately supports the loads. For a structure to be stable, the underlying materials must be adequately strong to support the loads from it.

Investigations on the capacity of geologic materials to support a proposed structure are carried out by direct methods such as borings and trial pits or through indirect geophysical methods such as resistivity and ground penetrating radar. Usually, it is impossible to define all subsoil characteristics through field exploration and laboratory testing. Combining information from various sites for the purposes of engineering design makes foundation designs safer.

The ancient foundation designs were based solely on precedent, intuition and common sense. To avoid a number of subsurface challenges, builders selected good sites for construction of structures. The builders knew that even the most carefully designed structures can fail if they are not supported by suitable foundations. Through trial and error, they developed rules for sizing and constructing foundations depending on the type of subsoil. The empirical rules usually produced acceptable results as long as they were applied to structures and soil conditions similar to those encountered in the past. However, the results were often disastrous when the builders extrapolated the rules to new conditions (Powell, 1884).

Records indicate that most areas in Nairobi city that are covered by alluvium and swamp soils were classified as unsuitable for foundation support during the early stages of city development (Sikes, 1934). However, with time, good sites posing no construction challenges became occupied and builders were forced to consider these sites with poorer soil conditions making foundation design and construction much more difficult. Loose soft soils often found beneath wetlands have been found to be closely associated with bearing capacity and settlement problems in some parts of the world (Seed, 1970; Bray *et al.*, 2004)

When the problematic soils also include organic material, they compress when loaded further aggravating the bearing capacity and settlement problems. Excessive settlement can lead to serviceability problems. Fenton *et al.* (2003) carried out studies on the effects of variability in soil properties on total and differential settlement of structural foundations. From the study, they concluded that unless the total settlements themselves are particularly large, it is actually differential settlements which lead to unsightly cracks in façades and structural elements, possibly even structural failure.

In seismic areas, loose saturated alluvium soils can become weak through the process of liquefaction. Moderate to strong ground shaking can create pore pressures in these soils, which temporarily decrease the shear strength. One of the most dramatic illustration of loss of strength in alluvial deposits occurred in Niigata, Japan, during the 1964 earthquake. Many of the buildings suddenly settled more than 1 m and these settlements were often accompanied by severe tilting, one apartment building tilted to an angle of 80° from the vertical! (Seed, 1970).

Numerous studies have indicated that the greatest element of risk in a building project lies within the uncertainties in ground conditions (Littlejohn *et al.*, 1994; Institution of Civil Engineers, 1991; National Research Council, 1984). The risks are significantly increased with inadequate geotechnical investigations resulting in unpredictable construction costs and programming (Collingwood, 2003). Consequently, site investigation forms a vital part of a building design, yet, in general the scope of such investigation is constrained by financial and time considerations. In reference to such constraints, the Institution of Civil Engineers (1991) stated that *"you pay for a site investigation whether you have one or not"*, suggesting that a limited site investigation will either result in gross over-designs or a foundation design that may not meet the design criteria. Additionally, the National Research Council (1984) concluded in a study of 89 underground projects that the level of geotechnical investigations in 85% of the cases was inadequate for accurate characterization. This inadequacy resulted in large cost overruns and time delays.

Site investigations aim to reduce uncertainty of ground conditions by various combinations of field and laboratory testing. Goldsworthy *et al.* (2004) carried out a study entitled "*Risk and reliability of site investigation*". They analyzed the performance of various site investigation schemes with respect to the cost of the resulting foundation system and the probability of failure.

An overdesign was defined as a conservative foundation design resulting from site investigation data being larger than the optimal. The designs based on inadequate site investigation were deemed not to comply. Goldsworthy *et al.* (2004) made the following conclusions:

i. A greater amount of information about a site will result in a foundation design that is less likely to fail. However the probability of overdesign shows a slight upward trend for site investigation of increasing scope.

ii. The magnitude of overdesign is much larger than the magnitude of underdesign for foundations on both low and high variability soils.

iii. A site investigation of a larger scope does not necessarily provide a less expensive foundation; however this is negated by the significant reduction in probability of failure.

The risk of foundation failure is heavily dependent on the quantity and quality of information obtained from a geotechnical site investigation aimed at characterizing the underlying soil conditions. Increasing the scope of the site investigation significantly reduces the risk of foundation failure, potentially saving clients and consultants large sums of money. To measure accurately the risk of a site investigation scope, it is necessary to attribute the costs resulting from decisions made regarding such an investigation.

Normally, the total cost of a structure is taken to include the cost of the site investigation, the cost of foundation construction designed from results of the investigation and the cost of building the superstructure. This approach usually leads designers to ignore or carry out insufficient site investigation to reduce the total cost of the structure. Goldsworthy *et al.* (2004) suggest that in addition to the above items, it is necessary to include costs of potential failures and refers to this as the *total cost approach.* In all, it is apparent that increased subsurface characterisation expenditure saves more than it costs.

To determine the potential failure cost, a failure severity scheme is used in three categories; minor retrofit; major retrofit; and demolish and rebuild as shown in Table 2.1. Day (1999) has provided settlement limits for each failure severity category on the basis of differential settlement as shown in Table 2.2.

Table 2.1: Failure Severity Scheme (Rawlinsons, 2002).

Failure severity	Failure description	Unit rate description
Minor	Some cracking evident from excessive settlement- requires patching and repainting	Minor refurbishment works divided by 2 (not include plumbing)
Major	Major cracking and structural failures – requires significant patching, structural retrofitting and foundation underpinning	Major refurbishment works + Foundation underpinning
Demolish and rebuild	Building can no longer be used for intended purposes- requires complete demolition and rebuild	Demolish costs + Rebuild costs

Table 2.2: Relationship between failure severity and differential settlement (Day, 1999)

Failure severity	Differential settlement (m) limits
No damage	25 mm
Minor	60 mm
Major	100 mm
Demolish and rebuild	130 mm

The deposits of Kerichwa Valley Series in Nairobi have the largest thickness along River Kerichwa and hence the name. The thickness of the series varies greatly as reported by Sikes (1934) and Morgan (1967). Areas underlain by deep KVS deposits such as Lavington Estate now have many high-rise residential blocks. Variable thickness of alluvial deposits beneath such structures can promote excessive differential settlement. For example, when part of a terrace of houses straddles an old river bed, that part is likely to settle by a different amount from the rest of the terrace.

Similarly, there is a risk of differential settlement occurring between a building which has been disturbed and neighbouring parts which have not e.g. adjoining buildings which may have finished their constructional settlement years ago. Richardson (2005) concluded that settlement in such structures can be difficult to control due to the constraints of the existing fabric. The remedial measures can only be carried out at the end of the settling period when the movements would have ceased (Kordahi, 2004). Richardson (2005) advised that provision should always be made within the project costs to pay for the monitoring and repair of any distress which may occur in structures supported on such deposits.

The Eiffel tower in Paris is an excellent example of a type of structure supported by subsoil with similar geological origin as Kerichwa Valley Series. It was originally built for Paris Universal Exposition in 1889 and was the tallest structure in the world. Alexandre Gustave Eiffel, the designer and builder, was very conscious of the need for adequate foundations, and clearly did not want to create another 'Leaning Tower of Pisa' (Kerisel, 1987). The Eiffel Tower was to be built at a site adjacent to Seine River filled with soft alluvial soils. Piers of Alma Bridge that were founded in this alluvium had settled by 1 m. The proposed tower could not tolerate such settlements. Using crude equipment of his time, Eiffel explored subsurface conditions of the site (Coduto, 2001). The studies revealed that the two legs of the tower closest to Seine River were underlain by softer and deeper alluvium, and were immediately adjacent to an old river channel that had filled with soft silt. Foundation design had to accommodate these soil conditions or else the two legs on the softer soils would settle more than the other two, causing the tower to tilt toward the river.

2.4 Shoring and underpinning

The increasing need for multilevel basements in built-up areas requires placement of foundations in excavations with vertical cuts. Challenges arise when such deep cuts have to be made in poorly graded soils adjacent to an existing structure (Ciancia *et al.*, 2006). In construction, shoring and underpinning operations are recommended when working at sites with weak materials.

2.4.1 Shoring

Shoring is the provision of support for existing structure while excavating underneath or adjacent to it. Shoring systems are installed for temporary and permanent earth retention. The systems also provide ground reinforcement and stabilization, speedy excavation and project schedule acceleration thus saving cost. Shoring is typically installed from the top down. A typical sequence would include installation of vertical structural members from the existing ground surface followed by the installation of anchors as excavation proceeds downwards. Figure 2.2 presents a shoring system installed to support a perimeter wall and pedestrian pavement at Hannover City, Germany. The vertical structural members in a shoring system resist lateral pressures and prevent soil ravelling. Anchors, consisting of steel tendons or high strength deformed bar, are also used to resist lateral pressures.

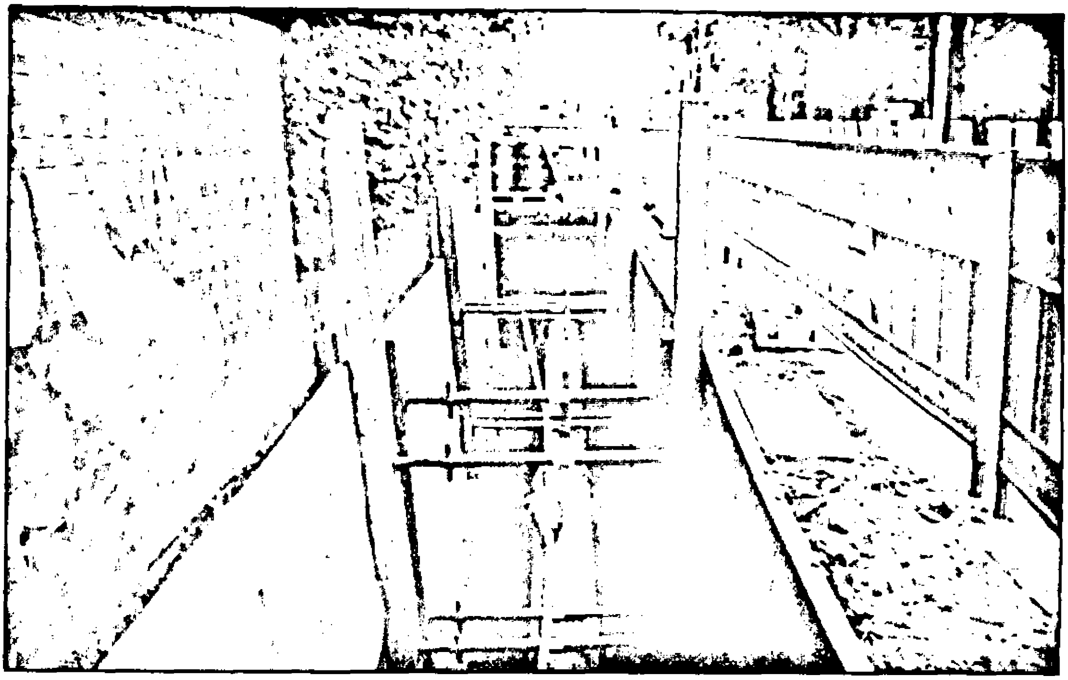

Figure 2.2: A shoring system installed to support a perimeter wall and pedestrian pavement at Hannover City, Germany.

Traditional shoring methods include timber shores (shown in Figure 2.2), sheet piling, soldier piles with lagging and tieback anchors (soil nailing and rock bolting). New methods utilize ground improvement systems such as soilcrete walls (formed by jet grouting or soil mixing techniques) and improved ground walls. Soil nail walls and reaction block anchor walls are installed as excavation proceeds downwards without the initial installation of vertical structural members. These shoring systems typically use shortcrete or precast blocks to control the ravelling of the excavation face during the performance of the work. Several methods of grouting can offer support, but only a few can resist compressive loads while also resisting lateral earth pressures. This is particularly so when historic and sensitive structures are in consideration and deformation tolerance is minimal (Burke, 2007).

Before selecting any shoring method one has to understand that support of existing structures, equipment and utilities requires means and methods that will lend this support without causing any harm. The successful installation of shoring is dependent on thorough and accurate characterization of site and subsurface conditions prior to design and installation. Important considerations in the installation of shores include:

i) The engineering characteristics of the soil and rock that will be encountered during shoring installation and the likelihood of variations in assumed subsurface conditions;

ii) The elevation and fluctuation of the ground water table;

iii) The location of existing structures, including utilities, and the estimation of the stresses induced upon the shoring systems from these structures;

iv) The allowable vertical and lateral movement that can be tolerated by existing structures (i.e. the required wall stiffness); and,

v) The need for temporary or permanent easements for wall anchor installation.

In a variety of circumstances, the use of deep mixing methods for the construction of excavation support systems is often the method of choice based on design requirements, site conditions/restraints and economics. Multiple auger deep mixing method was successfully utilized to limit lateral movement of adjacent structures, prevent the loss of support due to unravelling soils and control groundwater in a number of projects in Wisconsin and Pennsylvania by Andromalos and Bahner (2003).

2.4.2 Underpinning

Underpinning is the process of constructing or stabilizing the foundation of an existing building or other structure. Underpinning is accomplished by extending the foundation in depth or in breadth so it either rests on more supportive soil stratum or distributes its load across a greater area or by stabilizing the soil beneath the foundation. Underpinning may be necessary for a variety of reasons: the original foundation is simply not strong or stable enough; the usage of the structure has changed; the properties of the soil supporting the foundation may have changed (possibly through subsidence) or were mischaracterized during planning; the construction of nearby structures necessitates the excavation of soil supporting existing foundations; and, it is more economical, due to land price or otherwise, to work on the present structure's foundation than to build a new one. Underpinning can either be defined as remedial or precautionary. Remedial underpinning adds foundation capacity to an inadequately supported structure.

Precautionary underpinning is provided to obtain adequate foundation capacity to sustain higher loads or accommodate changes in ground conditions.

If underpinning is necessary to arrest settlement, it is essential that the underpinned foundations should be taken down to relatively unyielding ground below the zone of subsidence. Underpinning must be taken to a deeper and relatively incompressible stratum if necessary by means of piers or piles. In all cases where underpinning is provided close to excavations, it is important to design the underpinning members to carry any lateral loads transmitted to them from the retained earth or groundwater. The method of underpinning selected is dependent on the expertise required and the available equipment.

While underpinning Medical Research Institute facility in Ohio State University, Perko (2005) encountered very difficult soft soils and its location within an existing building and limited access for construction equipment complicated the construction process. The excavation measured 7 m by 13 m and was 4 m deep. The sides of the excavation could not be sloped due to spatial constraints within the existing building. Conventional shoring such as sheet pile or soldier pile walls was impossible due to low overhead clearance and limited areas to manoeuvre. In addition, the excavation bordered several masonry building walls with shallow footing foundations and an interior column. Other areas of the building remained occupied during construction, so vibration and noise were intolerable.

Many of the buildings that have gained landmark status in Nairobi City were built when there were no adjacent buildings. When new buildings are proposed adjacent to old buildings, tell-tale and datum points should be placed where necessary for observing the movement of cracks and settlements in structures adjacent to the excavation. Observations and measurements should be continued throughout the period of excavation, shoring or underpinning and until such a time thereafter as all detectable movements have ceased (Kordahi, 2004).

Figure 2.3 presents a structure supported on a reinforced slab foundation being underpinned at Sydney Australia while Figure 2.4 presents a structure along Luthuli Avenue at Nairobi City Centre whose foundation walls and footings were exposed during excavation and as a precaution should have been shored or underpinned to mitigate distress.

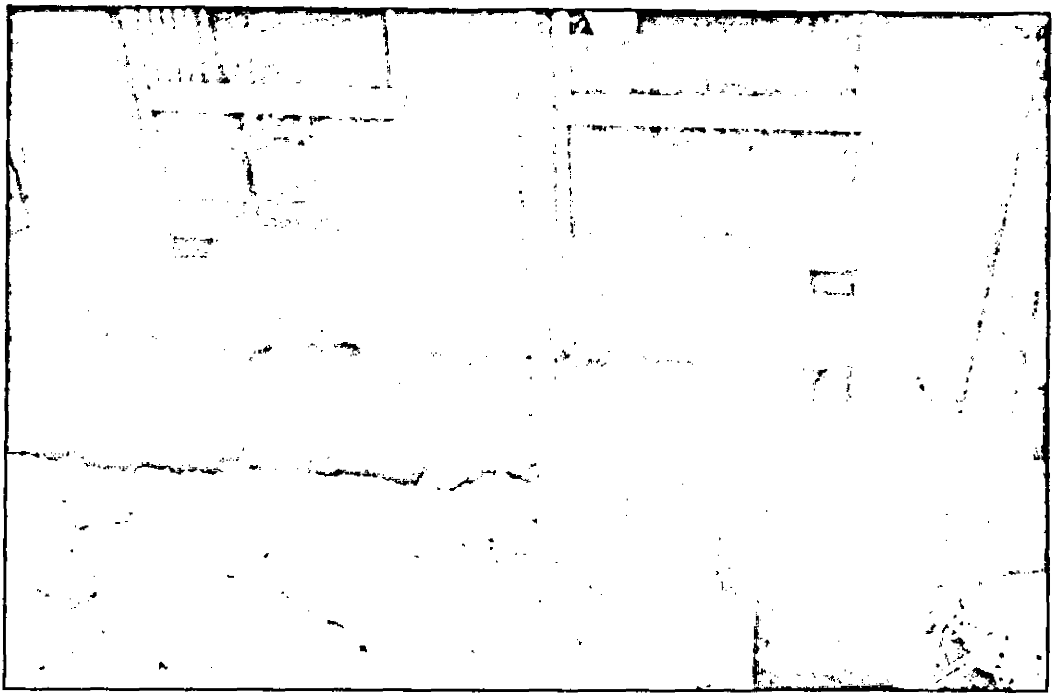

Figure 2.3: Underpinning of a building supported on a reinforced slab foundation at Sydney Australia (Therix Group, Australia)

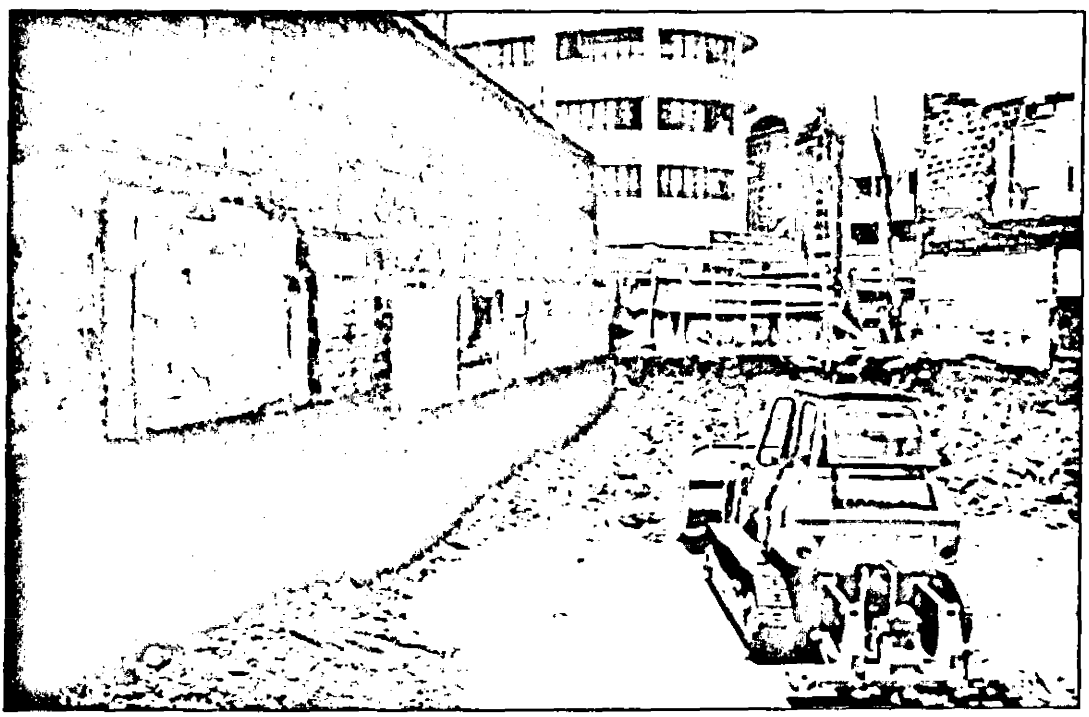

Figure 2.4: A structure along Luthuli Avenue at the City Centre whose foundation walls and footings were exposed during excavation and should have been shored

2.5 Gaps identified through literature review

The effects of extensive groundwater abstraction in major cities across the world have been documented as shown in the literature review. There are many localities in Nairobi City (Figure 2.1) that have a large concentration of boreholes and need to be investigated for effects of abstraction on rest levels. It is also apparent that consolidation settlement due to groundwater extraction can take many years before its effects are evident on the ground surface. Spatial and temporal groundwater rest level variations in Nairobi City were analysed from records of initial rest levels measured after borehole drilling. The best logged drillhole logs were selected to study the subsurface profile, aquifer and aquitard thicknesses and to estimate the consolidation settlement that can result from groundwater depletion in Nairobi City.

Information from contactors and consulting engineers indicates that poorly graded soils exist at various depths that are excavated when building basement floors in Nairobi City. Some structures supported on these soils have shown cracks attributable to effects of adjacent excavation. According to Tomlinson (1995) small settlements may not damage a structure or the cost of repairing damage and compensating the owner may be less than the cost of underpinning. This study has analysed the engineering characteristics of critical subsoil layers encountered at building sites and drill holes in Nairobi City to enable property developers to know circumstances under which they may be required to shore excavation sides or underpin existing foundations to avoid ground movement.

The records from City Council of Nairobi show that majority of low-rise and medium-rise structures in Nairobi City are designed without adequate information on foundation conditions. From the studies on subsurface characterisation, it can be observed that structures that are designed without knowledge of subsurface conditions cost more due to being overly conservative or are likely to fail (Collingwood, 2003; Littlejohn *et al.*, 1994; Goldsworthy *et al.*, 2004). It is necessary to determine the engineering properties of the subsurface and provide information can enable developers to design structures based on knowledge of subsurface conditions. Most cities in developed countries have such information available for their citizens. Jones (1981) explains that availing such scientific and technical information to citizens saves the country time and money required for emergency response for example due to collapse of a building.

Variable and difficult conditions have been encountered in construction sites across the city during construction of large buildings (Ministry of Works, 1969; Sikes, 1934; Gevaerts, 1964). Recent examples include the excavation for construction of Hazina Towers in the City Centre that caused distress to the adjacent buildings, large flows of groundwater capable of causing lateral pressures on basement walls beneath City Hall Annexe, and, sloping formations with groundwater flowing along the bedding plane. When geotechnical investigations on such high variability subsoil are not adequate, constructors are faced with a challenge of change in assumed structural and/or subsurface conditions revealed during excavation. Availing information on the subsurface conditions from various construction sites can provide additional information to what is revealed during construction and enable construction of better engineered structures.

The literature review indicates that settlement of structures starts during construction as the ground adjusts to the new weight imposed upon it. When structures are built on rocks, gravels or sands, constructional settlement is substantially complete by the end of construction. For clays, silts and peats however, settlement may take many years. Once constructional settlement is complete it will not recur, unless the status quo alters (Richardson, 1991). However, in Nairobi City, signs of distress have been noted in old residential buildings in Ngei, Garden and Madaraka Estates (Ministry of works unpublished reports). Analysis of the distribution and thickness of the loose soils across Nairobi City is necessary so as to indicate areas where long-term settlement is expected so that protection measures can be provided.

The Kerichwa Valley Series that provides support to most shallow to medium foundations in the city was designated by Sikes (1934) to include deep alluvium and agglomerate beds that owe their origin mainly to lacustrine deposition. Gevaerts (1964) recorded encounter of channels containing water during building operations and sealing of underground basements to large buildings that resulted in the diversion of water producing an additional hazard to the foundations of older buildings supported on KVS of limited loading capacity. Gevaerts (1964) further notes that, this substantially disturbed the balance between the ground and the structure and could promote new settlement. In the past, no extensive study has been done on the depths and localities with shallow groundwater leading to hazards during construction and in the life of the buildings. This study has characterised the various layers that comprise the KVS for behaviour during construction and under applied loads, especially when the water table is high.

CHAPTER THREE

METHODOLOGY

To meet the objectives of the study, both primary and secondary data were collected. The data collected were analyzed under two categories: geological investigations and geotechnical investigations. The various procedures in which these studies were carried out are discussed in this chapter.

3.1 Geological Investigations

3.1.1 Acquisition of secondary data

Secondary data was obtained from archival records and reports, publications and conference papers as well as from people who carried out work in the study area. The data includes;

i) Four topographical maps of scale 1:50,000 acquired from Survey of Kenya,

ii) Geological map of scale 1:125,000 acquired from the Ministry of Environment and Mineral Resources together with the geological report,

iii) Geological and hydrogeological reports obtained from the Ministry of Water and Irrigation.

iv) Research reports and journal/conference papers obtained from the Ministry of Education and University of Nairobi.

v) Rainfall data for the period 1975-2008 for 13 meteorological stations obtained from the Kenya Meteorological Department.

vi) Monthly rest level measurements for monitoring wells in the period 1971-1975 obtained from the Ministry of Water and Irrigation and for the 2006-2008 obtained from the Water Resources Management Authority (WRMA)

3.1.2 Acquisition of primary data

The site investigation phase of any geotechnical design whether by direct or indirect means plays a vital role, where inadequate characterization of the subsurface conditions may contribute to either a significantly over-designed solution or foundations that are likely to fail (Goldsworthy *et al.*, 2004). Projects with limited site investigation funds can carry out geophysical resistivity surveys after which boreholes can be drilled to verify the interpretation thus avoiding a significant amount of drilling. However, the amount of ground investigations required will vary among different types of projects and soil conditions.

Geophysical resistivity techniques are based on the response of the earth to the flow of electrical current. In these methods, an electrical current is passed through the ground using electrodes. Two potential electrodes allow recording of the resultant potential difference between the current electrodes, giving a way to measure the electrical impedance (ratio of potential to current) of the subsurface material. Because the earth is neither homogeneous nor isotropic, a measured voltage difference yields a resistivity value that is an average over the path along which the current follows. Data are thus termed 'apparent resistivity'. The apparent resistivity is a function of the measured impedance and the geometry of the electrode array.

Geological research prior to starting of the geophysical study identified a network of lineaments/ fractures in the south-western section of the project area which are predominantly trending north-northwest. The geophysical surveys were designed to relate the lineaments to the possible fractures as well as to map the stratigraphy of the subsurface. The preliminary study also involved identification of suitable sounding sites and seeking permission from relevant authorities to use the site on a specified day. The sounding centres were selected based on the lineaments that had been geologically identified particularly for the purpose of bridging the gap in knowledge about the subsurface at the locality by relating the nearest borehole log information to the geoelectric properties, and, on the availability of space for carrying out geophysical resistivity survey.

Vertical electrical sounding was carried out at forty eight (48) sites with current electrode distances ranging from 80 m to 500 m. Many of the sounding centres were located in public institutions or along straight portions of roads. A number of sounding centres were also located adjacent or near boreholes with known well log data so as to determine the correlation between the geophysical attributes and the hydrogeological and mechanical properties of the subsurface. Each sounding centre constituted a vertical sounding (VS) and designated VS-1 through to VS-48 as shown in Figure 3.1. The sites are labelled along south-north lines in the west-east direction beginning with VS-1 through VS-48. The geological units on this map are as explained in Figure 1.3.

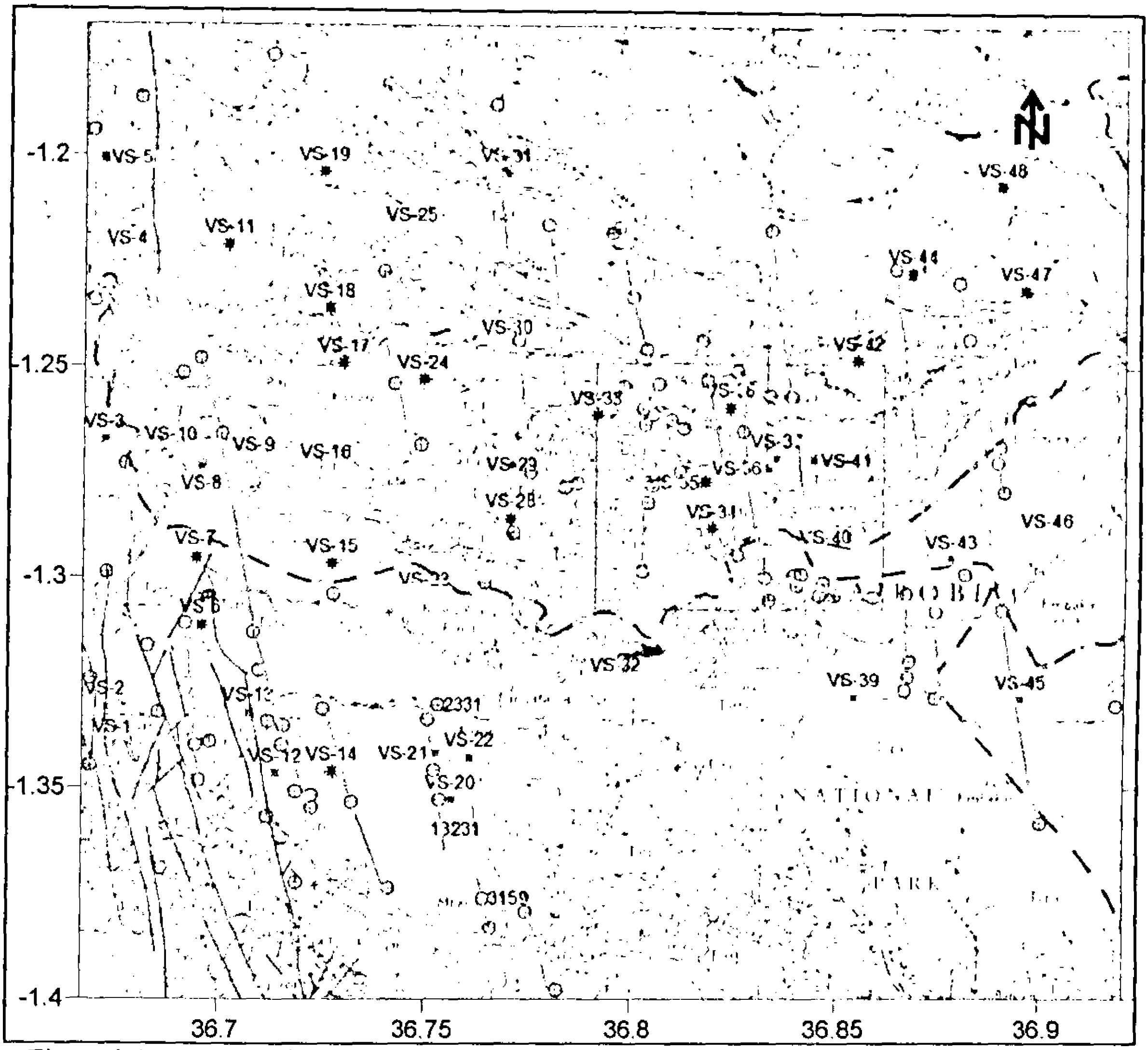

Figure 3.1: Map showing vertical electrical sounding sites (stars), positions of boreholes with fracture traces (purple circles), fractures traced (green and pink) and outline of the City Centre and immediate environs (red rectangle) (map after Saggerson, 1991)

The geophysical resistivity survey was conducted using a WDDS-1 Digital Resistivity Meter manufactured by Benteng Digital Control Technology Institute in China. WDDS-1 resistivity meter is a new generation of resistivity instrument that can automatically measure and store parameters such as self potential, current, voltage, geometric factor and apparent resistivity. The equipment basically consists of three parts, i.e. Central Processing Unit (CPU), transmitting unit and a receiving unit. Voltage signal is input through the potential pots, transformed by impedance and deduced by self potential (SP), filtered and then finally transformed. The current signal is sampled by standard impedance and then isolated, filtered, amplified and transformed. CPU unit functions by taking out the transformed voltage and current signals and then transmitting them to display once the execution is complete. All calculations are carried out by the WDDS-1 resistivity meter and saved to be downloaded to a computer.

The Schlumberger array was utilized to carry out the resistivity survey. A nested electrode configuration with internal spacing of $\frac{MN}{2}$ for the potential electrodes and an increased distance of $\frac{AB}{2}$ from the centre for the current electrodes as shown in Figure 3.2 was used. The potential electrodes were fixed at one location while the current electrodes were expanded about the centre point. Only when the current electrodes became relatively distant did the potential electrode spacing need to be expanded in order to have measurable potentials. Schlumberger technique was preferred because it provides for high signal-to-noise ratios, good resolution of horizontal layers, and good depth sensitivity (Ward, 1990).

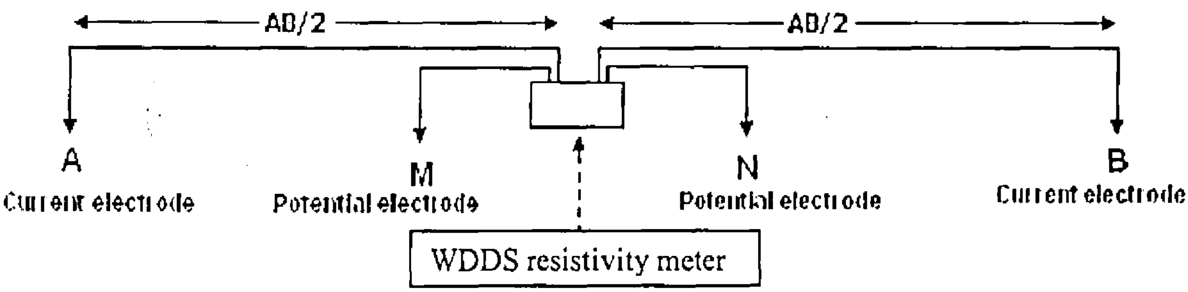

Figure 3.2: Schlumberger resistivity arrangement (Ward, 1990)

3.1.3 Analysis of geological data

3.1.2.1 Analysis of hydrogeological parameters

Borehole depths as well as hydrogeological parameters such as borehole yields, drawdown and recovery were studied from core logs and hydrogeological reports. The hydrogeological parameters were plotted as contours on Surfer (a software package developed by Golden Software of Golden, Colorado) in order to compare their variation across the study area.

Rest level hydrographs were plotted from monthly groundwater levels measured at 21 monitoring wells for the period August 2006 to March 2008 and for 1971 to 1975 period. This was to express the rest level variations in one dimension. The water level hydrographs are presented in Appendix 1. The regular monthly water level measurements obtained for the period 1971-1975 were commenced as a response to public concern regarding rapid water level drop at various localities. Before the period of monitoring, the largest water level drop of 100 m was recorded at Langata Barracks at Nairobi National Park. Ruaraka and Karen also experienced receding water levels whereby most old boreholes run dry or the yields were too uneconomical to abstract. Another water level monitoring regime began in 2006 after the formation of WRMA but the steam seems to have reduced by 2008 as no records of rest levels for the succeeding period were obtained.

The apparent resistivity values obtained for each survey line were plotted against half the current electrode spacing (AB/2) on a log-log plot. Two different softwares i.e. Interpex 1-D sounding and Earth Imager were used to characterise the subsurface. Interpex 1-D software is suitable for detecting aquifers whereas the Earth Imager software has the capability of recognizing thinner subsurface layers in addition to detecting aquifers. The simulated profiles and the corresponding resistivities were used to identify water-bearing zones in the subsurface and to provide a geologic context for evaluating the results of geotechnical investigations. The aquifers identified on the Interpex 1-D software as well as their thickness are presented in Appendix 2 while results for the other analyses are discussed in Chapter Four.

3.1.2.2 Analysis of water rest level variations on GIS software

Scanned and geo-referenced topographical sheets in raster format were digitized and vectorized on screen using GIS software to derive the relevant features. Surface drainage and contours were prepared as base maps in the analysis of temporal and spatial variation in groundwater rest levels using GIS. The contours were used in the analysis of the relation between topography and water rest levels while the drainage map was used to analyse the relationship between borehole yields, rest levels and surface water sources.

Hydrogeological data for 1600 boreholes as well as logs for 900 boreholes were obtained from MW&I. Five hundred boreholes had no coordinates. The missing coordinates for boreholes manually plotted on base maps were established by pointing a cursor on the georeferenced map and recording the data after which it was keyed in on Microsoft Excel files. Coordinates for all boreholes were converted to comma separated values (CVS) after which they were transferred to Notepad. Using the data in Notepad, the borehole positions were plotted in GIS environment to express the distribution of boreholes in the study area.

Due to the fact that the boreholes are not evenly distributed and that no continuous measurement of groundwater rest levels for all boreholes exists, a simple method was devised whereby the boreholes were grouped into decades to visualize initial water level variations. To analyse temporal rest level variations, measurements starting from late 1920s and ending in 2010 were divided into seven decades. This was because boreholes for the 1920s and 1930s as well as 1960s and 1970s were few and were thus combined. Water rest level in each borehole was considered as a value and compared with the surrounding for analysis of spatial variation.

In the absence of precise altitude measurements at borehole positions, an Advanced Space-borne Thermal Emission and Reflection radiometer (ASTER) 30 m planar resolution Digital Terrain Model (DTM) was used as terrain reference. This DTM is geo-referenced to the WGS84 geoid with an estimated vertical accuracy of 8 m and estimated horizontal accuracy of 12 m both at 68% confidence. The location of each borehole was registered to the DTM for purposes of calculating the precise altitude above sea level and for spatial modelling.

Since borehole positions and DTM had different resolutions, an interpolation was carried out in order to calculate the precise altitude. Interpolation in the vicinity of a borehole was carried out assuming that the groundwater rest level between neighbouring boreholes is continuous and smooth. Kriging method was selected for interpolation because of the sparse or uneven nature of data points. Kriging is a geostatistical interpolation method based on the assumption of spatial autocorrelation. The correlation states that the distance and direction between sample points are the major factors governing the estimated values at unknown points (Gold, 1989). The weights values are calculated by taking into account spatial structure of data distribution represented by a sample variogram derived by the distances between boreholes.

After interpolation by Kriging method, groundwater "surfaces" were generated for each of the seven decades. Because of the difference in elevation between the east and the west, it was easy to identify some boreholes that were plotted in wrong locations because they showed significant peaks above the surrounding. A filtering process was carried out by identifying the boreholes represented by peaks and counterchecking with the information in records and positions plotted on base maps after drilling. Approximately 10% of data points with significant peaks were found to be plotted at different locations on base maps and GIS. It was difficult to establish which of the two points represented the true position on the ground and the data points were thus eliminated.

Rainfall hydrographs were plotted for monthly and annual totals for a period of 32 years (1975-2008). Shapes of the graphs were used in understanding rainfall distribution and intensity across the year and how they are related to water rest levels in boreholes and groundwater recharge. They were also used to explain some peaks in water level surfaces on GIS. The data used was for the following stations: Ngong Forest Station, Ngong Divisional Office, National Agricultural Laboratories, Kigwa Estate; Kiambu, Mathari Hospital, Moi Airbase, Muguga K.A.R.I, Runda Water Ltd, Wilson Airport, Dagoretti Corner, Kikuyu Agricultural Office, Jomo Kenyatta International Airport and Kabete Agromet.

Other significant peaks were found to be related to the method of reporting locations (by contours or GPS) that could result in varying the location of data point by 100 m to 300 m. Boreholes 300 m apart could be found close together resulting in peaks on 3-D surfaces. After omitting or rectifying the data points affected by the above-mentioned factors, modelling of groundwater surfaces for all epochs and regeneration of spatial models was done to represent the variation in water rest levels through the various decades.

3.1.2.3 Analysis of water rest level variations on Surfer software

To establish historic variation in the shape of the groundwater surfaces in two dimensions, water rest levels in relation to sea level for the various decades were plotted as contours on Surfer. The information required for plotting the Surfer contours included borehole identity, coordinates and water rest level. This information was already filtered during the GIS analysis where many of the erroneous data was eliminated. The trends in the Surfer contours enabled identification of the changes in rest levels over the 80-year period.

3.1.2. 4 Analysis of subsurface profiles and aquifer geometry

For analysis of the stratigraphy, a preliminary quality assurance was carried out on the raw data for 900 boreholes that had been drilled by different companies and over a long period of time. Discrepancy in interpretations of even the same stratum and the way in which borehole locations were recorded was evident. All poorly logged wells were not used in the analysis. To eliminate problems that could be related to borehole location, data points that plotted on positions in disagreement with location name in records were rectified where possible or eliminated. Boreholes without rest level record were also eliminated. In the end, 457 borehole logs were found suitable for use in the analysis of stratigraphy.

The general subsurface profile as well as the thickness and aerial extents of the various geological units were analysed from the 457 drillhole logs. The positions of the boreholes were plotted on the map so as to ensure that all the geological units covering the study area were represented in the profiles. To harmonise the different interpretations of the same rock type by different drillers, entire borehole log profiles were typed in worksheets. The profile descriptions were then interpreted in reference to 29 cross sections prepared by Gevaerts (1964).

Each drill hole log consisted of the following information: borehole identity (BHID); geographic coordinates (x,y); thickness of the stratum (from-to); the name of the geologic unit (formation) or hydrogeologic unit identity (HGUID) as well as description of the formation and code (HGUcode). Subsurface profiles for the individual boreholes were plotted using Strater software (by Golden Software) and compared at several locations across the area to provide a sight into the subsurface stratigraphy.

The aquifer geometry of the study area was studied from 307 best logged boreholes. The confined system was divided into aquifers, aquitards and hydraulic separators. The aquitards are old land surface and Athi Series clays while the hydraulic separators are the rocks. Although the method of reporting aquifers differs from driller to driller, the aquifer thicknesses were calculated for all the boreholes. The thickness contours were plotted on surfer to express the aquifer geometry across the study area.

3.1.2.5 Analysis of probable settlement due to groundwater withdrawal

Aquifer properties such as transmissivity, storativity and specific storage were calculated from test pumping records. These properties together with aquifer and aquitard thicknesses were used in the analysis of settlement that can result from groundwater depletion. The calculations were based on the theory of compaction due to groundwater withdrawal as proposed by Poland *et al.* (1972). Various other formulae used in settlement analysis are discussed here.

In test pumping, extracting water from a well at a known rate causes drawdown. Once pumping is stopped, well and aquifer water levels rise towards their pre-pumping levels. The rate at which this occurs depends on many factors including aquifer thickness and permeability. Transmissivity (T) is a measure of the amount of water that can be transmitted horizontally through a unit width by the full saturated thickness of the aquifer under a hydraulic gradient of 1. Transmissivity of aquifers is calculated using Equation 3.1 (Batte *et al.*, 2008):

$$T = \left\{ \frac{2.3Q}{4\pi\Delta s} \right\} \qquad (3.1)$$

Where,

Δs = Change in the head (m),

Q = final borehole yield (m^3/ hr).

Transmissivity could also be obtained from plotting drawdown/recovery versus log time graph and finding the slope of the graph at a point towards the end of the recovery period. Values obtained from analysis of the recovery record serve to countercheck calculations based on the pumping records (Fletcher, 1989).

Storativity (S) is the volume of water that a permeable unit will absorb or expel from storage per unit surface area per unit change in head. Storativity is a dimensionless quantity. Specific storage (S_s) is the amount of water per unit volume of a saturated formation that is stored or expelled from storage owing to compressibility of the mineral skeleton and the pore water per unit change in head (also called the "elastic storage coefficient"). It is caused by pressure changes and the resulting expansion and contraction of the aquifer and the groundwater.

Groundwater withdrawal results in fluid pressure change in the subsoil. If the pressure head in a confined aquifer reduces, the effective stress acting on the aquifer will increase; the aquifer will consolidate due to this increased stress. The hydraulic head drop in the aquifer will eventually result in the same amount of head drop in the confining layer (aquitard) as well (Sun, 1997). In response to this drainage, the effective stress will increase, resulting in a commensurate volume reduction in the aquitard itself.

Consolidation due to groundwater withdrawal comprises both elastic and inelastic changes. The elastic consolidation of an aquifer refers to the compaction that can be recovered after the induced stress is lifted. This consolidation of the confined aquifer is related to its storativity and compressibility. Consider a homogeneous, isotropic and horizontal confined aquifer; assume the stress increment producing the abnormal fluid pressure was vertical only and the individual solid component making up the rock is incompressible. From Jacob (1940),

$$S_s = \rho_w g (\beta_p + n \beta_w) \tag{3.2}$$

Where,

S_s = the specific storage,

ρ_w = the density of water,

g = gravitational acceleration,

β_p = the bulk compressibility of the rock,

n = the porosity of the aquifer,

β_w - is the compressibility of the water (4.6×10^{-10} m²/N).

Storativity is the product of specific storage and aquifer thickness (Pucci *et al.*, 1994):

$$S = S_s m = \rho_w g m (\beta_p + n \beta_w) \tag{3.3}$$

Where,

m = thickness of the aquifer

By definition (Domenico, 1983) rock bulk compressibility is given by:

$$\frac{\Delta m}{m} = \beta_p \Delta \sigma = \beta_p \Delta P \qquad (3.4)$$

Where,

Δm = layer thickness change

$\Delta\sigma$ = effective stress change = fluid pressure change, ΔP

Combining Eq 3.3 and 3.4 and solving for Δm gives the elastic subsidence change of a confined aquifer

$$\Delta m = \Delta P \left(\frac{S}{\rho wg} - nm\,\beta_w\right) \qquad (3.5)$$

Inelastic compaction refers to the compaction that cannot be recovered after the stress is lifted. It is mainly occurs in the aquitards (Sun, 1997). The maximum subsidence due to inelastic compaction is given by Domenico and Schwartz (1998) as:

$$\Delta m' = S_s' \, m \, \frac{(\Delta h_1 + \Delta h_2)}{2} \qquad (3.6)$$

Where,

$\Delta m'$ = the thickness change of the aquitard,

S_s = the specific storage of the aquitard,

m = the thickness of the confined aquifer,

Δh_1 and Δh_2 = the head change in the upper and lower confining beds.

After analysing the evidence of general and specific drop in rest levels and determining the average aquifer thicknesses, probable settlement due to groundwater withdrawal was calculated using Equations 3.2 to 3.6. Porosity of the aquifers was assumed as 0.25 (Sun *et al.*, 1999). The transmissivity and storativity values used in settlement analysis are presented in Appendix 3. The probable settlement due to consolidation of clay layers was calculated for 23 sites representing various subsurface profiles across the study area and the results are presented in Chapter Four.

In some localities such as Industrial Area, Karen, Langata, Ruaraka and Lower Kabete, storativity values from more than one borehole were used for settlement analysis. Calculation of storativity for other localities such as Parklands and Westlands data was limited to only one borehole because of test pumping records. The storativity values used here were calculated from values of drawdown during pumping and the thickness of aquifers indicated on the core logs. More accurate storativity can only be determined by measurement of an observation well response during the constant rate pumping of a production well (Van Tonder *et al.*, 1998).

3.1.2.6 Analysis of fracture traces

Fracture traces are linear features on the earth's surface that are naturally occurring and are surface manifestations of subsurface fracture zones in the bedrock. When viewed in cross-section, fracture traces are seen to be vertical or near vertical breaks in the bedrock. Surface water bodies, such as streams or rivers meander along paths of least resistance. An abrupt straightening of a river or stream is a good indication that it is following the path of least resistance created by an underlying fracture zone. Lines of increased moisture may be seen as vegetation alignments. For example, a tree species which requires more water than is normally available may appear along a fracture zone. Similarly, greater water availability can result in a local increase in vegetation size and density.

Fracture zones are identified in borehole logs because deeply weathered rock associated with fracture zones causes numerous drilling problems including caving conditions, loss of drilling fluids and difficulty with drill bit rotation. Other indicators include abnormally large unconsolidated zone thickness, high moisture content in the unconsolidated zone, highly weathered bedrock, presence of iron staining and authigenic minerals (such as calcite) on fractured rock surfaces, and increases in water yield with depth.

The purpose of fracture trace analysis in this study was three-fold:

1. To analyze structural jointing and fracture trends and their possible association with abrupt changes in soil type at construction sites and borehole yields,
2. To determine weak areas that support structures, and
3. To assist in evaluating if fracturing and jointing trends observed on the surface are present at depth within on-site boreholes.

900 borehole logs were analysed for fracture traces of which 97 had one or more of the indicators of fracturing. The levels of abrupt change in rock type as well as collapsible zones were recorded to be compared with the fracture trace depths recorded in boreholes outside the

fault zone. The summary of fracture traces in boreholes is presented in Appendix 4. Positions of the boreholes with fracture trace indicators were plotted on the geological map using Didger software (by Golden Software, Colorado) and those aligned were taken to represent fractures as shown in Figure 3.1.

In addition to the usual indicators of fracture traces, the artesian wells at Kangemi High School, Braeburn School and Muthurwa were also taken to present traces. This was based on the following observations by Hove (1973): faults affect not only the water storage, but also the pattern of its migration and consequent discharge in Nairobi Area; several springs issue along faults in the north-west; if a stream flows parallel to a fault plane in the high ground, groundwater recharge takes place, discharge (springs) occurs into the stream in the lower ground.

3.2. Geotechnical investigations

The scope of this subsection was to investigate the geotechnical character of the subsurface geological materials and features that are likely to have or have had considerable engineering implications on civil engineering structures in Nairobi City.

3.2.1 Acquisition of Secondary data

Drilling boreholes for geotechnical site investigation is a very expensive exercise only carried out for large construction projects. It could have been impossible for such information to be collected for the purpose this study. However, enormous amount of data is available (459 boreholes and 644 trial pits), thanks to failure investigation and site investigation reports for 183 sites archived at the Materials Testing and Research Department at the Ministry of Roads as well as records for permit applications submitted to the City Council of Nairobi (1914 – 2008).

The sites from which geotechnical data was obtained are indicated in Figure 3.3. The geological units on this map are as explained in Figure 1.3. The investigation sites are concentrated at the City Centre because it is where most of the high-rise structures are located. The raw data for each of the sites investigated is available in records. It is indicated that the soil samples collected during the investigations were tested for classification according to the Unified Soil Classification System (1957). The rocks were tested according to International Society of Rock

Mechanics (ISRM) standards reviewed from time to time. The soil classification tests include: moisture content, Atterberg limits, particle size distribution, consolidation and in situ density. The soil and rock strength tests include triaxial and direct shear, unconfined compressive strength and point load strength tests. The principles behind the geotechnical tests carried out and application of the test results are briefly explained below. The results of the analysis of the raw data obtained during the study are presented in Chapter Four.

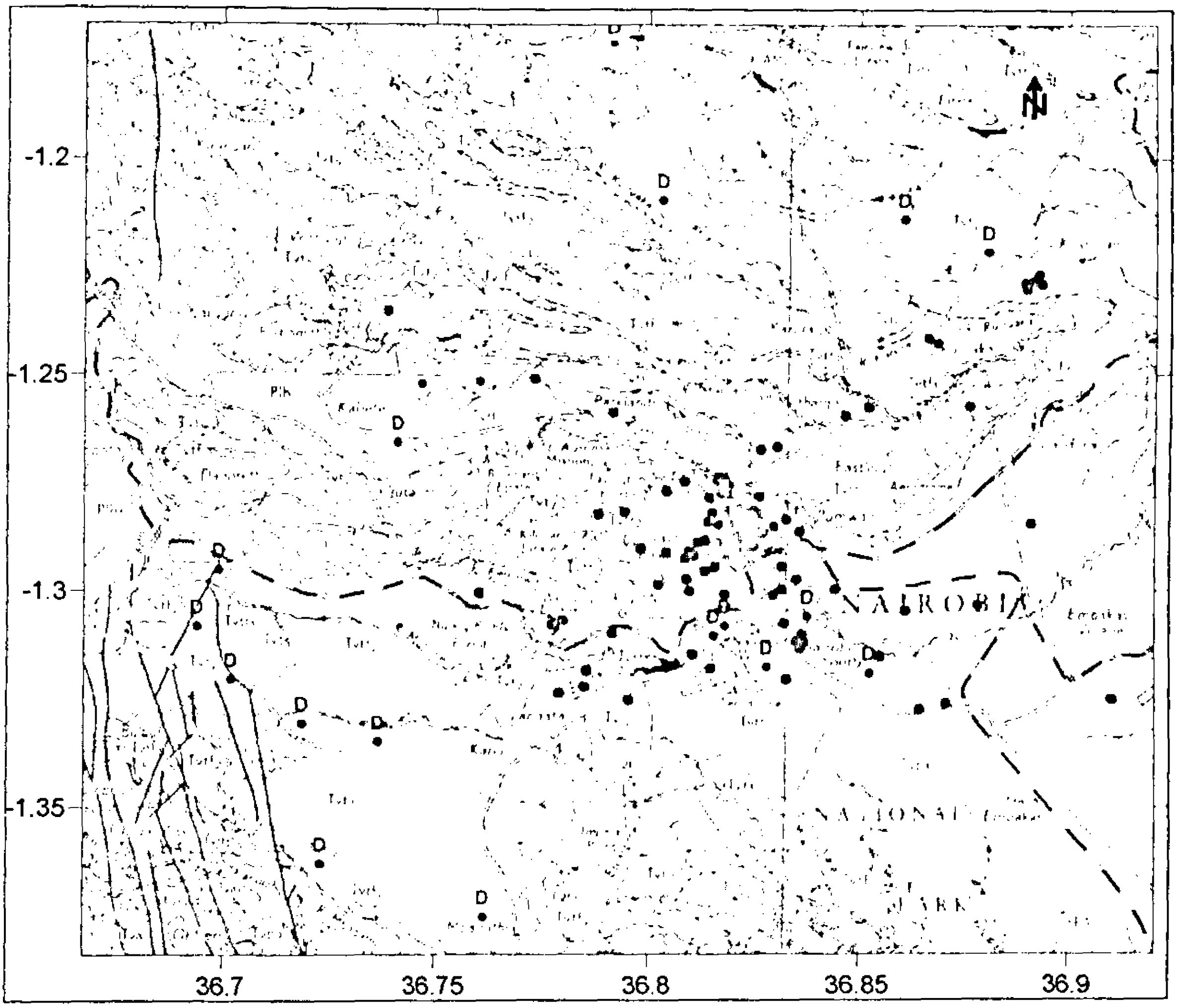

Figure 3.3: Map showing sites for geotechnical investigation (red circles), distress in structures (blue circles) and sample collection (light blue stars)

3.2.1.1 Moisture content

Moisture content is the quantity of water in a mass of soil, expressed in percentage by weight of water in the mass. Moisture content of natural soils will depend on their degree of saturation but can vary considerably from less than 5% for dry sand to several 100% for montmorillonite clay or peat. The natural moisture content test is carried out as an index test.

3.2.1.2 Atterberg limits

Atterberg limits are the limits of water content used to define engineering soil behaviour. Liquid limit (LL) is the lowest water content above which soil behaves like liquid, normally below 100%. Plastic limit (PL) is the lowest water content at which soil behaves like a plastic material, normally below 40%. Shrinkage limit (SL) refers to the point at which further reduction in moisture content does not cause reduction in volume. A low shrinkage limit indicates that a soil will begin to swell at low water content. If natural moisture content lies above the shrinkage limit, then the soil will shrink on drying. The range of water content within which a soil behaves as a plastic material is known as plastic range. Plasticity index, PI, is the numerical difference between plastic limit and liquid limit.

3.2.1.3 Particle size analysis

The particle size analysis test has the scope of grouping constituent particles into separate ranges of sizes. The ultimate purpose is to determine the relative proportions of dry weight of each size range especially the clay fraction (colloid content) that has direct bearing on the engineering properties of soils (Terzaghi *et al.*, 1996).

3.2.1.4 Triaxial shear and direct shear tests

The purpose of triaxial testing is to determine strength characteristics of soils and the variation of strength with lateral confinement, pore water pressures, drainage and consolidation. Direct shear test provides an alternative method of evaluating the shear strength of soil. The test involves applying a load to a split ring device containing the soil sample. The strength is determined based on maximum resistance to loading. Either reconstituted or intact samples of cohesionless or cohesive soil can be used. There are various limitations associated with direct shear device. These include boundary conditions during testing, imposed failure surface and the inability to control drainage during testing. Often the direct shear test is viewed as an index test for strength whose application will vary from location to location.

3.2.1.5 Consolidation tests

When loads (e.g. embankments, spread footings) are placed or constructed on soils, the soils compress. The compression can be very rapid as in the case of granular soils, or very slow as in the case of cohesive soils. The calculation of settlement involves many factors including the magnitude of the load, change in strength at the depths, permeability of the soil, water table location and stress history for the soil. Consolidation testing is performed to determine the effects of these factors on the amount and rate of soil compressibility. One dimensional consolidation tests are performed in an oedometer (Terzaghi, 1943). This device consists of a confining ring that contains the soil sample and an apparatus for applying a vertical load on a loading platen located on top of the soil specimen.

During the test, increments of the vertical static load are applied to the undrained soil sample while recording the corresponding settlement. The consolidation test results include the presentation of stress-void ratio in a semi-logarithmic scale. From the changes in thickness at the end of each loading step, one can determine the compression index (C_c) and coefficient of volume compressibility (m_v). The coefficient of consolidation (c_v) and the rate of consolidation can be also measured using the results of the thickness changes of the sample against time during a load step.

The consolidation settlement is computed using the Terzaghi method considering the thickness of unconsolidated deposits, overburden pressure and structural loads. According to Terzaghi (1936), when a soil stratum of initial thickness H_0 has fully consolidated under a pressure increment $\Delta 6'$, the final consolidation settlement ρ_f can be computed as:

$$\rho_f = m_v H_0 . \Delta\sigma' \qquad (3.7)$$

Assuming the pressure increment $\Delta 6'$ is transmitted uniformly over the thickness H_0.

When the compressible soil layer is thick, vertical stress due to finite surface loading decreases non-linearly with depth. Hence, Equation 3.7 is applied for computing the settlement of a thin layer dz and then integrated over the total thickness H

$$\Delta\rho_f = m_v \Delta\sigma . dz \qquad (3.8)$$

$$\Delta\rho_f = m_v H_0 \Delta\sigma' \qquad (3.9)$$

Both m_v and $\Delta\sigma'$ are variables.

The integration may be performed numerically by dividing the total thickness H into a convenient number of layers and computing the settlement for each layer taking $\Delta\sigma'$ at the middle of a layer as representing the average pressure increment for that layer. Another equation for computing settlement is given below:

$$\rho_f = \frac{C_c}{1+e_0} H \log_{10} \frac{\sigma_0' + \Delta\sigma'}{\sigma_0'} \qquad (3.10)$$

Where,

H = initial thickness of consolidating layer

e_0 = initial void ratio of the layer

C_c = compression index

σ_0' = initial effective pressure at the middle of layer

$\Delta\sigma'$ = effective initial increment at the middle of the layer

3.2.1.6 Swell potential of clays

This soil test is not carried out routinely on all samples but is done when the soils physically look suspicious or originate from materials that are known to have unusual engineering properties. The one dimensional swell potential test is used to determine the percentage swell and swell pressures developed by swelling soils.

Swelling is a characteristic reaction of some clays to saturation. The potential swell depends on mineralogical composition. While montmorillonite exhibits high swell potential, illite exhibits none to moderate swell characteristics and kaolinite has almost none. The percentage of volumetric swell depends on the amount of clay, its relative density, permeability, and location of the water table, presence of vegetation and trees and overburden stress. Percentage of volumetric swell is investigated by the free swell test.

3.2.1.7 Collapse potential of soils

The collapse potential test is similar to a consolidation test. The sample is placed in a consolidation system and an axial load is applied while the sample is dry. The sample is then flooded and vertical compression of the sample recorded with time. At high moisture content, these soils collapse and undergo sudden changes in volume. The collapse during wetting occurs due to destruction of clay or calcium carbonate binding which provides the original strength of these soils.

3.2.2 Acquisition of primary data

3.2.2.1 Investigation of distress in structures

Investigation of distress in structures began by a study of records at City Engineer's Department where information on areas with sensitive soils and distress in structures was obtained. Distressed structures that were still standing were investigated by visual inspection and photography. Eyewitness accounts were recorded to gain information about the design and construction of the structures, and any events that occurred afterwards. Geotechnical data from the vicinity of the distressed structures was used for identifying the strata, determining the physical properties, analyzing the bearing capacity and allowable soil pressures of failed materials.

3.2.2.2 Geotechnical investigations at drilling sites

The Ministry of Roads provided information on dates of drilling for geotechnical investigations at flyover sites on the Nairobi- Thika Road, and bridge sites on the Eastern and Northern bypasses. In liaison with the drilling teams, logging was carried out on freshly cored samples. The site log included description of the physical characteristics of the subsoil, groundwater levels and levels of strata changes. Records of total core recovery (TCR), rock quality designation (RQD) and fracture index (FI) for each run of the core barrel from rotary drilling were made. A thorough study of geological reports was carried out after which description of the formation was amended where necessary in the light of information from past geological survey as well as laboratory testing.

3.2.2.3 Investigation of construction excavations

The sites where excavations were investigated are summarised in Table 3.1. Excavation sides were visually inspected and the general information concerning location of boundaries between hard materials and overlying soft deposits, groundwater conditions and liability to ground movement was recorded and compared with existing information. In all cases samples could not be collected for classification because none of the structures were supported on loose soil and the excavated soil was too disturbed to provide any meaningful results for building design. Also, the Development Control Department at the Nairobi City Council gets information on structures that are already underway.

Table 3.1: Sites where excavations were investigated and information collected

Site location	Excavation depth (m)
Chiromo Road	14
National Water Conservation and Pipeline Corporation, Industrial Area	5
Acacia Centre, Hurlingham	4
Kongoni Primary School, South C	5
Strathmore University	6
Opposite Ramogi studio, Luthuli Avenue	4
Catholic Church, Tena Estate	3
Parklands	6
Jumuia Place, Lenana Road	4
Cedar Springs, Riara Road	4

3.2.2.4 Investigation of construction difficulties and shoring and/or underpinning operations

The City Council provided information on consulting engineers and contractors that have overseen certain projects within the city. Telephone contacts and physical addresses of the consulting firms were obtained from the plans submitted to the City Council. Questionnaires were sent to 68 consulting engineers requesting for information on some specific projects carried out. The information requested for in each questionnaire included: name of structure, type of structure, part of the city, number of storeys, foundation conditions, groundwater conditions, method of shoring and underpinning of adjacent structures (if any) and level of success. A sample of the questionnaire used is as shown in Appendix 5.

Follow-up was made by way of calls to request for appointment. Thirty nine consulting engineers offered invitation and provided valuable information on the foundation problems, causes of distress in structures and the methods of coping with the problems as well as any shoring/underpinning operations carried out at 85 sites. On-going projects at the level of foundation were visited in the company of the consultant engineers. Three consultants filled in the questionnaires, attached the reports and sent by post.

3.2.3 Analysis of geotechnical data

3.2.3.1 Soil classification

Plotting the results of liquid limit and plasticity index test on a Casagrande plasticity chart enables the various soil types to be classified in order of compressibility. The higher the liquid limit and plasticity index, the higher the compressibility. Plasticity charts for some of the geotechnical investigation sites are presented in Appendix 6 while a summary of the results is presented in form of a table in Chapter Four.

Atterberg limits are also used to predict liquefaction potential of soils. Liquefaction occurs when loose saturated sediments are subjected to ground vibrations of greater than 0.2 g resulting in total or substantial loss of shear strength. During liquefaction, the affected soil behaves as a liquid or semi-viscous substance and can cause structural distress or failure due to ground settlement, loss of bearing capacity in foundation soils and the buoyant rise of buried structures. (Seed, 1970).

Field and laboratory studies show that the occurrence of liquefaction depends on many factors, such as earthquake magnitude, shaking duration, peak ground motion (acceleration/velocity), depth to the groundwater table, basin structures, site effects and liquefaction susceptibility of sediments (Wang *et al.*, 2006). The study was based on updated compilation of worldwide data to provide a new relation between the liquefaction limit and earthquake magnitude. Galli (2000) also reported a linear relation between magnitude (M) and the distance (R_{max}) within which liquefaction may occur for liquefaction in Italy from both historical and instrumental records. The results of these studies are plotted in Figure 3.4. These results are generally consistent with the understanding that the intensity of ground motion increases with earthquake magnitude and attenuates with epicentral distance.

Data on the frequency and magnitude of earthquakes that affect the city was obtained from the University of Nairobi and Wanyumba (2001). The data was plotted on Figure 3.4 based on the subsoil characteristics and the distance of the site from Lake Magadi in Kenya that is known to be the most seismically active part of the Rift Valley. This was to investigate whether the loose deposits that underlie Nairobi are liable to liquefaction and determine any relation between earthquakes and defects in structures.

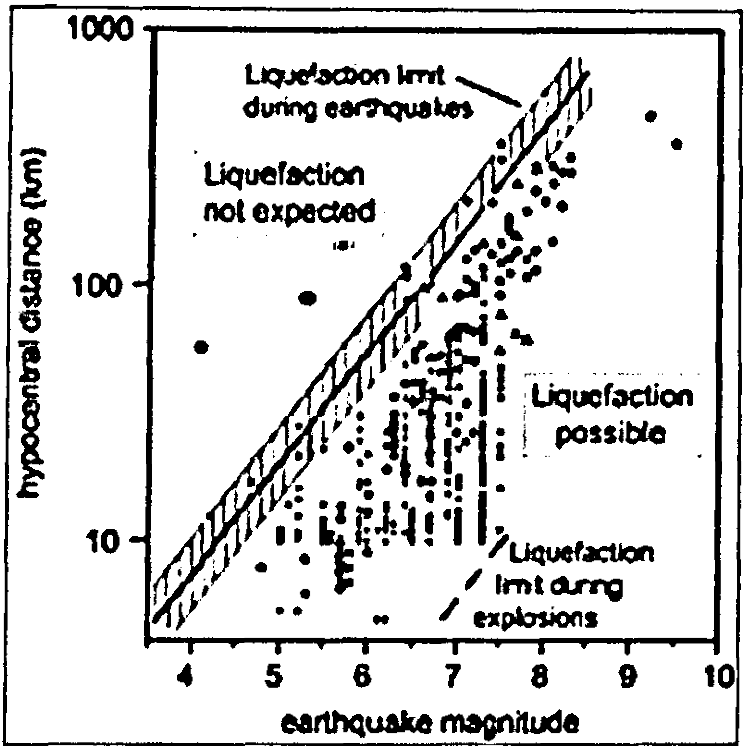

Figure 3.4: Chart for determining potential of liquefaction (Wang et al., 2006)

(OR- Ongata Rongai, NC- Nairobi City centre)

From the particle size analyses, points of cumulative percentage quantities finer than certain sizes (e.g. passing a given size sieve mesh) were plotted against log size. A smooth S-shaped curve drawn through these points is called a grading curve. The position and shape of the grading curve determines the soil class. Unified Soil Classification System (USCS) was used to classify the soils for engineering purpose by combining particle size characteristics, liquid limit and plasticity index. A summary of the information obtained from grading curves is presented in a table in Chapter Four.

The raw data from triaxial shear and direct shear tests was plotted on graphs of normal stress against shear stress to obtain values of cohesion and internal friction from which shear strength was calculated according to the Mohr- Coulomb criterion (Ranjan and Rao, 2002). The strength was then defined in terms of friction angle and cohesion for total and effective stress conditions or in terms of undrained shear strength as presented in a table in Chapter Four.

The consolidation that occurs when the soil specimen is wetted without any additional load is called potential hydrocollapse strain ($\mathcal{E}_w$). Potential hydrocollapse strain calculated from the raw data was used to classify soils according to the criterion given by Coduto (2001) as shown in Table 3.2.

Table 3.2: Qualitative assessment of collapsible soils (after Coduto, 2001)

potential hydrocollapse strain $\mathcal{E}_w$	Severity of the problem
0- 0.01	No problem
0.01-0.05	Moderate trouble
0.05-0.10	Trouble
0.1-0.20	Severe trouble
>0.20	Very severe trouble

The required remedial measures are specified based on qualitative assessment of the test results e.g. soil with a strain $\mathcal{E}_w$ greater than 0.1 must always be excavated and compacted back in place. However the qualitative assessment does not provide an estimate of the potential settlement due to hydrocollapse. For example a thick stratum of "moderate trouble" soil that becomes wetted at a great depth may cause more settlement than a "severe trouble" soil that is either thin or does not become wetted to as great depth.

3.2.3.2 Bearing capacity calculation

The database used for bearing capacity analysis has been compiled over 60 years approximately, and represents sites with high-rise buildings and areas with sensitive soil. Site investigation has mainly been carried out by wash boring of overburden followed by rotary drilling. Boreholes for multi-storeyed buildings at the City Centre are terminated at predetermined depth regardless of the subsoil conditions. The number of geotechnical boreholes drilled at a site is determined by the developer and their depths range between 10-28 m. Soil and rock samples collected are tested for compressive strength used for calculating bearing capacity. There are two main methods for establishing rock strength: uniaxial compressive strength and point load strength tests.

Uniaxial compressive strength (UCS)

Uniaxial compression test is the most direct method for determining rock strength. The laboratory uniaxial compressive strength tests are carried out on rock cores without physical fractures (intact rocks) after soaking in water for seven days. Depending on the length of core the test is carried out either axially or diametrally on the specimen.

The Cylindrical rock specimens are tested in compression without lateral confinement. When the specimens are too soft, the confinement can be provided as in a triaxial test. If strain for each step during compression is measured, elastic modulus of intact rock can be obtained by this method. Also, by including lateral strain measurements in the test, it is possible to determine the Poisson's ratio of the test specimen. Since strains in rocks are usually very small, the accuracy and resolution of the strain measuring instrument must be very high.

Some layers penetrated by geotechnical boreholes in Nairobi City are described as highly fractured or decomposed. A highly fractured rock mass is one that contains two or more discontinuity sets with typical joint spacing that is small with respect to the foundation width. Highly fractured or decomposed rock behaves in a manner similar to dense cohesionless sands and gravels when loaded. As such, the mode of failure is likely to be general shear. Knowledge of the depth, type and size of foundations is important in the study of bearing capacity of foundations resting on rock.

When ground formations are loaded in situ, the vertical loading will always induce lateral subgrade reaction due to lateral confinement. By considering crushing of rock under the footing and with subgrade reaction acting equal to compressive strength (q_u), Goodman (1989) suggested the following equation for a case where the rock is heavily fractured:

$$q_{ult} = q_u (N_\Phi + 1) \qquad (3.11)$$

Where,

q_{ult} = the ultimate bearing capacity

q_u = the ultimate stress (uniaxial compressive strength)

$N_\Phi = \tan^2 (45 + \frac{\phi}{2})$

φ = friction angle of intact rock

It is important to note that the q_u values used for bearing capacity calculation are for one-dimensional loading conditions and therefore they represent failure values at extremely rare and worst site conditions. Hence in such circumstances, the failure stress will be much higher (twice as much as q_u). As per Equation 3.11, the q_{ult} will generally be high for rock mass. The actual values will be lower mainly due to the rotation and sliding of some blocks within the zone of influence. With the uncertainty involved in the estimation of q_u and Φ it is always desirable to adopt larger factor of safety or conservative values of q_u and Φ (Sowers, 1979).

It has been found that a rupture surface may develop on one side of a foundation due to defects in the rock (Ramamurthy, 2008). Additionally, in cases where the shear failure is likely to develop along planes of discontinuity or through highly fractured rock masses such as those underlying Nairobi, cohesion cannot be relied upon to provide resistance to failure. The allowable bearing pressure value is determined by applying a suitable factor of safety to the calculated ultimate bearing capacity. Foundations located above the water table will develop significantly more resistance to potential bearing capacity failures than foundations below the water table.

Point load strength

The purpose of point load strength testing is to determine the strength classification of rock materials. This test provides an indication of strength much more quickly and inexpensively than uniaxial compression testing. However, the strength obtained from this test is not as accurate as an unconfined compression test. Therefore the test results are used as an index of strength and generally not used directly for design purposes. The test can be performed in the field with portable equipment or in the laboratory. In the test, a cylindrical sample is trimmed and seated on the apparatus after which it is loaded to failure. The load at failure is divided by the cross-sectional area to obtain normal stress value.

Using the Point load test results, the bearing capacity is computed by relating the values of stress obtained to the uniaxial compressive strength (q_u) of the rock by Equation 3.12 (Goodman, 1989).

$$q_u = K\, Is_{50}, \qquad (3.12)$$

Where,

q_u = the ultimate stress (uniaxial compressive strength)

Is_{50} = the stress value obtained from the point load test.

The value of K could vary from 15-35 (ISRM, 1985) for most rocks; often it is taken in the range of 20-25. Some weathered sedimentary rocks like sandstones, siltstones and shaley mudstones have given K values as low as 6, beyond the range suggested by ISRM (Ramamurthy, 2008). The allowable bearing pressure value is determined by applying a suitable factor of safety to the calculated ultimate bearing capacity. Foundations located above the water table will develop significantly more resistance to potential bearing capacity failures than foundations below the water table.

For engineering use, rocks are classified based on individual property such as UCS, modulus, sonic velocity and point load strength index. When no test data for q_u and Φ is available, RQD from bore log may be adopted with caution to estimate the ultimate bearing capacity from Ramamurthy (2008):

$$q_{ult} = q_u \left(\frac{RQD}{100}\right)^2 \qquad (3.13)$$

Table 3.3 presents the classification for intact rocks based on the UCS as proposed by ISRM (1981) whereas Table 3.4 presents rock mass classification based on rock quality designation (RQD) as proposed by Deere (1964)

Table 3.3: Uniaxial compressive strength classification of intact rocks (ISRM, 1981)

Class	**UCS (MPa)**	**Strength description**
1	<6	Very low strength
2	6-20	Low strength
3	20-60	Moderate strength
4	60-200	High strength
5	>200	Very high strength

Table 3.4: Classification of rock mass based on RQD (Deere, 1964)

RQD (%)	Rock quality
0-25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

3.2.3.3 Correlation of resistivity and mechanical properties

For areas without past geotechnical tests, correlations between the resistivity values and the UCS were used to determine the character of the subsurface. The equation for relating resistivity and UCS was derived from two experiments (Keller, 1974; Kate and Sthapak, 1995) that depicted a curvilinear increase in UCS with resistivity. Keller performed experiment on saturated quartz monzonites while Kate and Sthapak experimented on 26 samples of igneous, sedimentary and metamorphic origin. A generalized relationship between UCS (MPa) and ρ (ohm-m) is expressed in the form of Equation 3.14 (Ramamurthy, 2008).

$$\log_{10} q_u = 0.2 \log_{10}\rho + 1.2 \qquad (3.14)$$

Deshmukh and Gokhale (1990) studied the variation of point load strength and resistivity for saturated coalmine and observed linear increase in point load strength with increase in ρ. The variation of punching shear strength τ_p with electrical resistivity of rocks obtained by Gokhale (2000) depicted a trend of linear increase in τ_p with increase in ρ on log scale. The generalised relationship between punching shear strength and ρ (ohm-m) is given as follows:

$$\tau_p = 19.55 \log_{10} \rho - 68.9 \qquad (3.15)$$

Keller (1974) also reported curvilinear increase in resistivity with increase in Young's Modulus on log-log plot for thermally soaked Palaeocene basalt. Donaldson (1975) reported decrease in compressibility and increase in modulus of elasticity with increase in resistivity for monzonite. Kate and Rao (1989) reported curvilinear increase in resistivity with increase in both the tangent modulus of elasticity as well as Poisson's ratio for dry sandstones. In all cases, high resistivity signifies superior engineering properties.

A comparison of mechanical properties and resistivity data was done for sites adjacent to boreholes. Results for five VES sites are presented in Appendix 7 while the figures are presented in Chapter Four. This comparison takes into account the individual resistivity values at each depth and relates this to the borehole log. The assumption in this criterion is that when very close resistivity values are obtained for successive depths of penetration, the strength properties of the layers are the same. Change in resistivity values with successive depth of penetration indicates change in mechanical properties of the subsoil and hence a different layer. When approaching groundwater, resistivity continues to reduce with greater depth until it gets to a minimum and then starts to rise.

3.2.3.8 Foundation designs used in Nairobi City

An extensive database surpassing one hundred and fifty sites supporting low-rise, medium-rise and high-rise structures was reviewed and summarised as shown in Appendix 8, Appendix 9 and Appendix 10. Information from Nairobi City Engineer and consulting engineers indicates that design of low-rise buildings is in most cases preceded by inspection of trial pits dug to sound material. In some cases, information on design bearing pressure and bearing stratum is obtained from a private firm called Soils Laboratories or determined on site (D.O.S). In general, conservative soil and rock shear strength parameters are adopted for design of foundations to take care of any form of settlement due to the physical weight of the buildings.

The main types of foundations used to support structural loads include footings (pads) that are provided to support columns of a building frame, raft foundation that can be considered as a large footing extending over a great area, frequently an entire building area, to support columns and walls of a building and pile foundations that are driven or otherwise into the soil to carry vertical compression load from buildings, bridges etc or to resist horizontal or inclined loads by retaining wall, bridge pier, waterfront structures and structures subjected to wind and seismic force. A pile can be a column of wood, concrete, reinforced concrete or steel.

Raft foundation is used on soils with low bearing capacity or where soil conditions are variable and erratic. Figure 3.5 shows a raft foundation that was built to support a classroom at Kongoni Primary School while Figure 3.6 is shows an expression of a pile foundation supporting a mast.

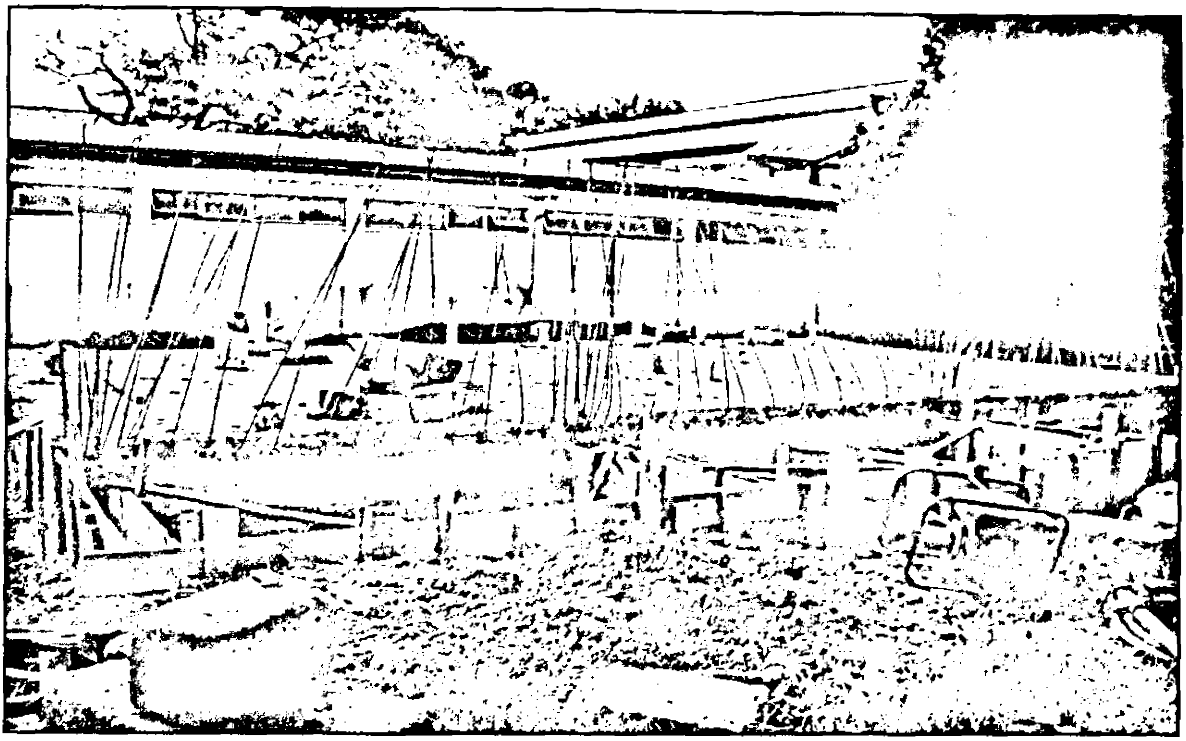

Figure 3.5: A raft foundation that was built to support a classroom at Kongoni Primary School.

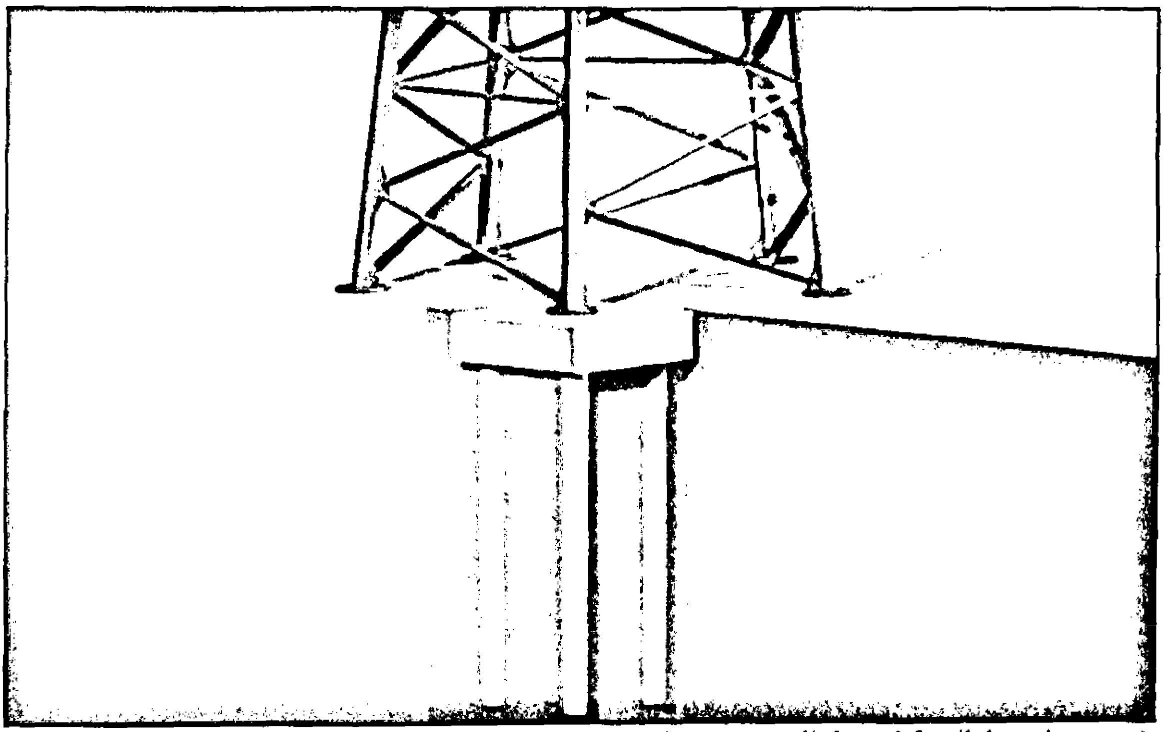

Figure 3.6: An expression of a pile foundation supporting a mast (littlerockfamilyhousing.com)

CHAPTER FOUR

PRESENTATION OF RESULTS

4.1 Geological conditions

4.1.1 Variation of hydrogeological parameters in space and time

Many boreholes across the city have been drilled in response to increase in population and increase in industrial activities. The first boreholes were drilled at Ruaraka area, mainly to supply industries and farms in that area. Other areas with old boreholes are Lower Kabete and Karen-Langata where the water was also used for farming. Intensive abstraction of water in Nairobi's Industrial Area started in the early 1980s after the curfew imposed on drilling boreholes was lifted. Currently, the largest concentration of boreholes per square kilometre is in the Industrial Area, followed by Karen-Langata area as shown on the density and distribution map (Figure 2.1). Because of the temporal and spatial variation of hydrogeological parameters, the six main abstraction areas, that is, Karen, Langata, Industrial Area, City Centre, Lower Kabete and Ruaraka are indicated on the Surfer contour maps for reference.

Depths to which boreholes in Nairobi are drilled vary from time to time, place to place and also according to the individual needs of the owner. Currently, the depths generally range between 200 m and 340 m. Figure 4.1 is a contour map of borehole depths for all decades based on data from 1299 boreholes. From the map, it can be noted that most of the deep boreholes are located to the west of the City Centre. It is also noteworthy that in the entire study area, newer boreholes are generally deeper than the adjacent old ones. This can be attributed to the requirement to seal upper aquifers to avoid depletion of adjacent boreholes and also to the desire to tap more water. Another reason for borehole termination is the encounter of sticky clays of the Lower Athi Series that makes drilling difficult. These clays occur at 200-250 m depth in East Nairobi area and Embakasi. In a number of boreholes, the clays appear to act as good aquitards as most of the aquifers encountered above them have high yielding wells.

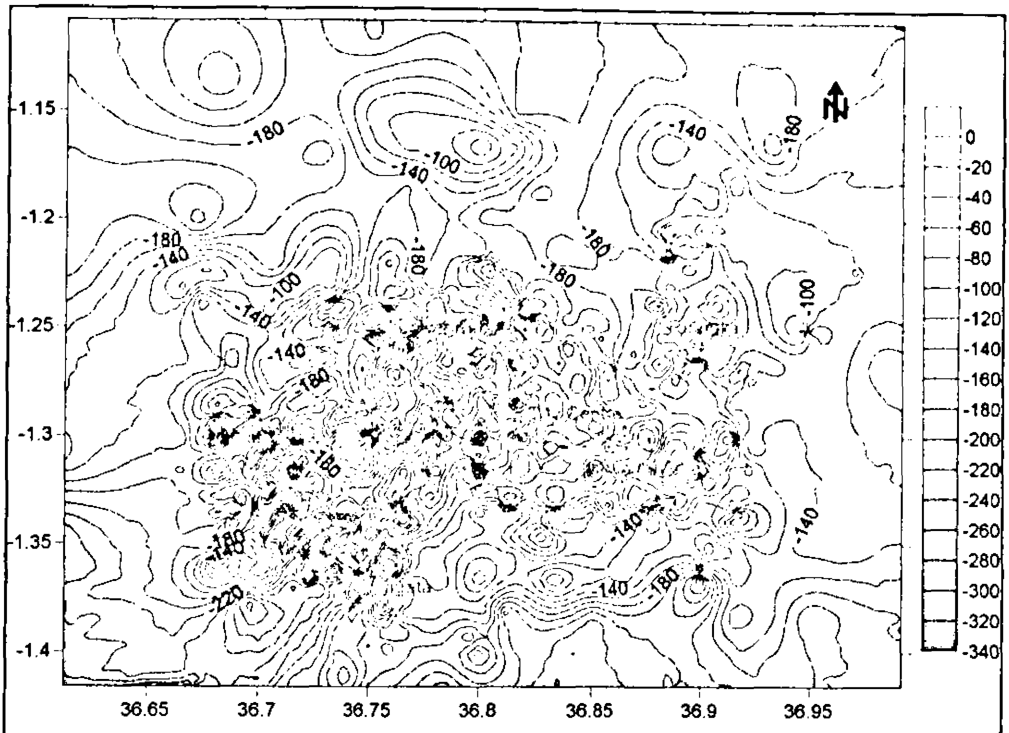

Figure 4.1: Contours of borehole depths

Figure 4.2 presents contours of average borehole yield in cubic metres per hour during pumping. The yields range from 58 m^3/hr in one borehole in Ruaraka to very low yields (<1 m^3/hr) obtained in boreholes in Karen and Langata areas. Boreholes in Ruaraka area are known to have high yields when they are initially drilled but the yields drop drastically after a few years in operation. Examples are those found at Kahawa Barracks and East African Breweries. The average yield all over the study area as indicated by the frequency of the contours is 6 m^3/hr. The high yielding borehole at City Centre is BH No. C-1987 at Uhuru Highway railway crossing that is thought to penetrate a fault.

On Figure 4.2, it can also be observed that the borehole yields in Karen and Langata areas are mainly low to moderate (< 10 m^3/hr). This is probably because of the small thickness of the Athi Series (50-70 m) that yields much of the groundwater in the rest of the study area. In other localities, the Athi Series is thicker; for example, it is 305 m thick at the City Centre while at Kasarani it is about 140 m. Boreholes in Industrial Area and Embakasi have higher yields and experience less drawdown owing to this thickness.

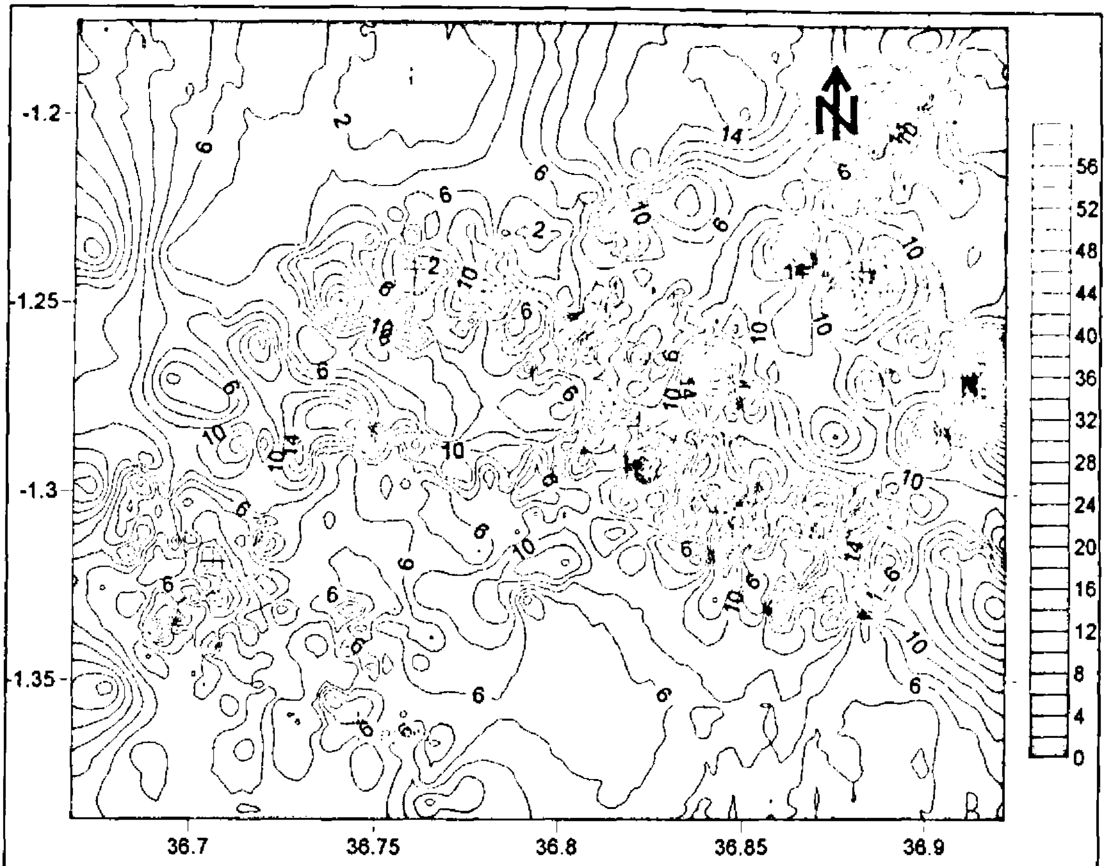

Figure 4.2: Contour map showing borehole yields in cubic metres per hour

Water rest levels in boreholes in Nairobi City are measured from the ground surface using a dip meter and reported in metres. The water rest level is influenced by the type of main aquifer struck; whether unconfined or confined. When an unconfined aquifer is encountered, groundwater rests at the level of the aquifer (representing the water table) and when it is a confined aquifer, the rest level rises above the level of the first aquifer (representing the pressure surface). In some instances, water level rests below the first water strike; this is so when the first aquifer is minor. At Kangemi High School, Braeburn Secondary School in Garden Estate and at Muthurwa, the water levels rose to the ground surface when the main aquifers were struck in the boreholes. Such boreholes are referred to as artesian. Figure 4.3 presents contours of drawdown while Figure 4.4 presents contours of recovery during test pumping.

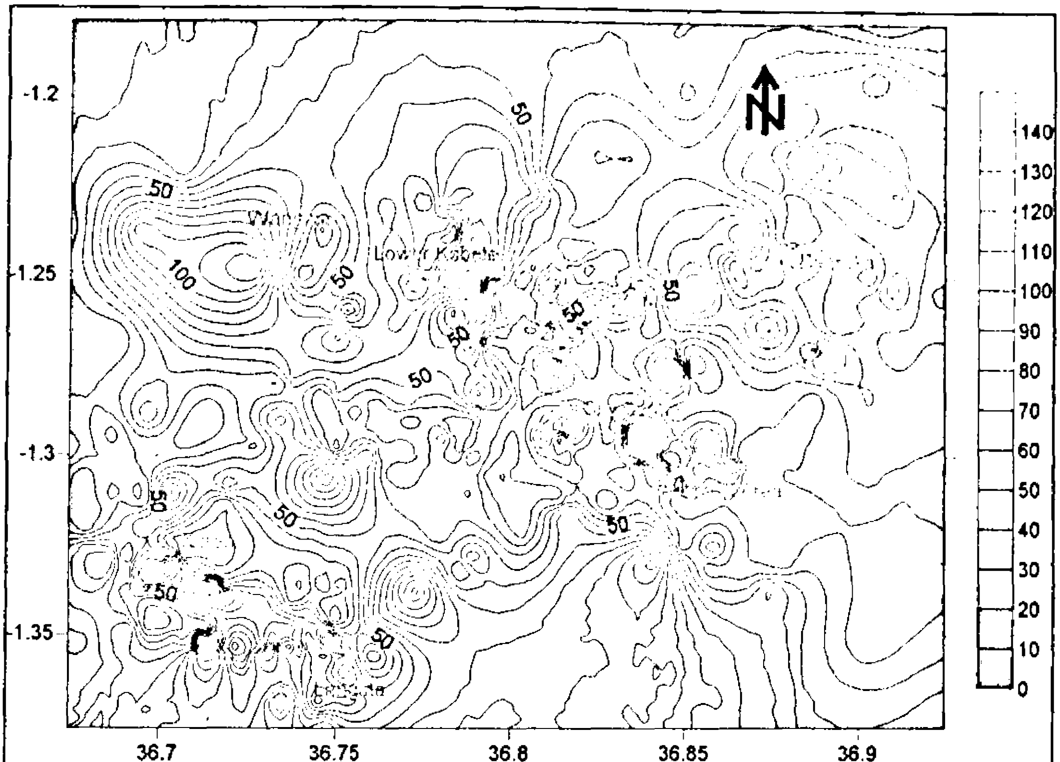

Figure 4.3: Contour map of drawdown in metres during pumping

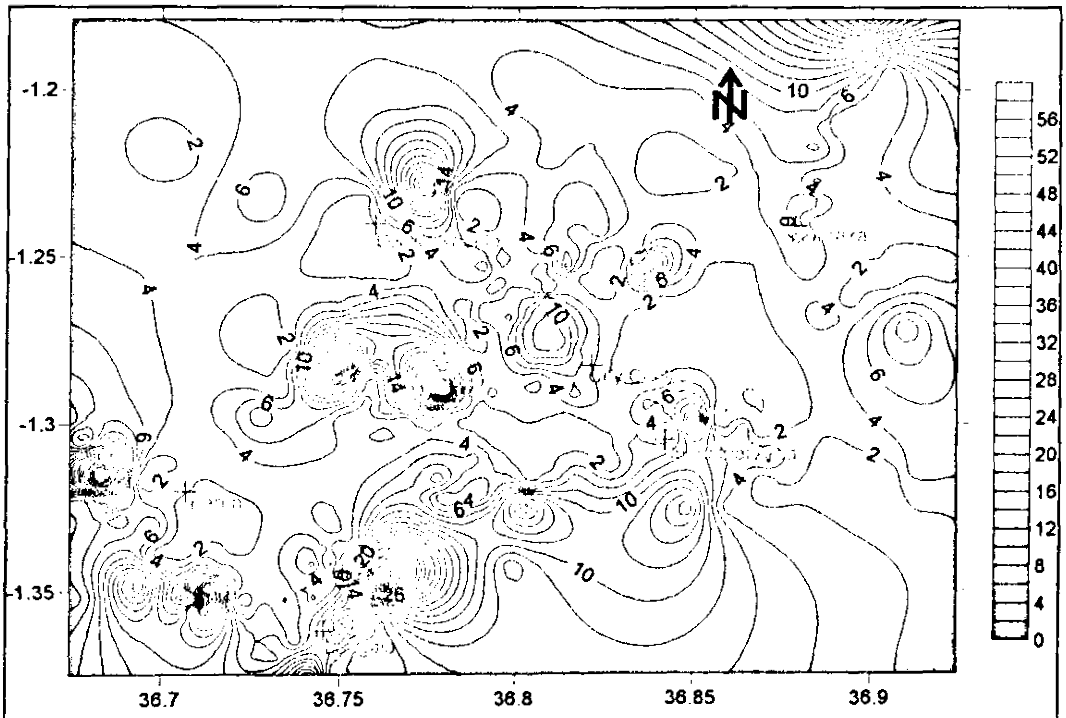

Figure 4.4: Contour map of water level recovery in metres after 24 hours pumping

Records indicate that a drawdown of 140 m was experienced during test pumping of a borehole at Wangige. The borehole recovered only 58 m of the drawdown. Complete recovery was not attained in the borehole and hence there was a residual drawdown of about 82 m after the initial test pumping. Test pumping of BH C-2373 at Langata Barracks lead to a drawdown of 128 m and a recovery of 5 m, resulting in a residual drawdown of 123 m. Continuous rest level measurements in 1971-1975 indicate that the borehole had a further recovery of 23 m. Large initial drawdown is also observed in BH C-2413 along Lower Kabete Road at Spring Valley that experienced a drawdown of 100 m and a recovery of 4 m; a residual drawdown of 96 m.

4.1.2 Variation of water rest level as analysed on GIS

Figure 4.5 depicts an image of rest levels superimposed on the DTM of the study area. The height difference between the west and the east is 650 m.

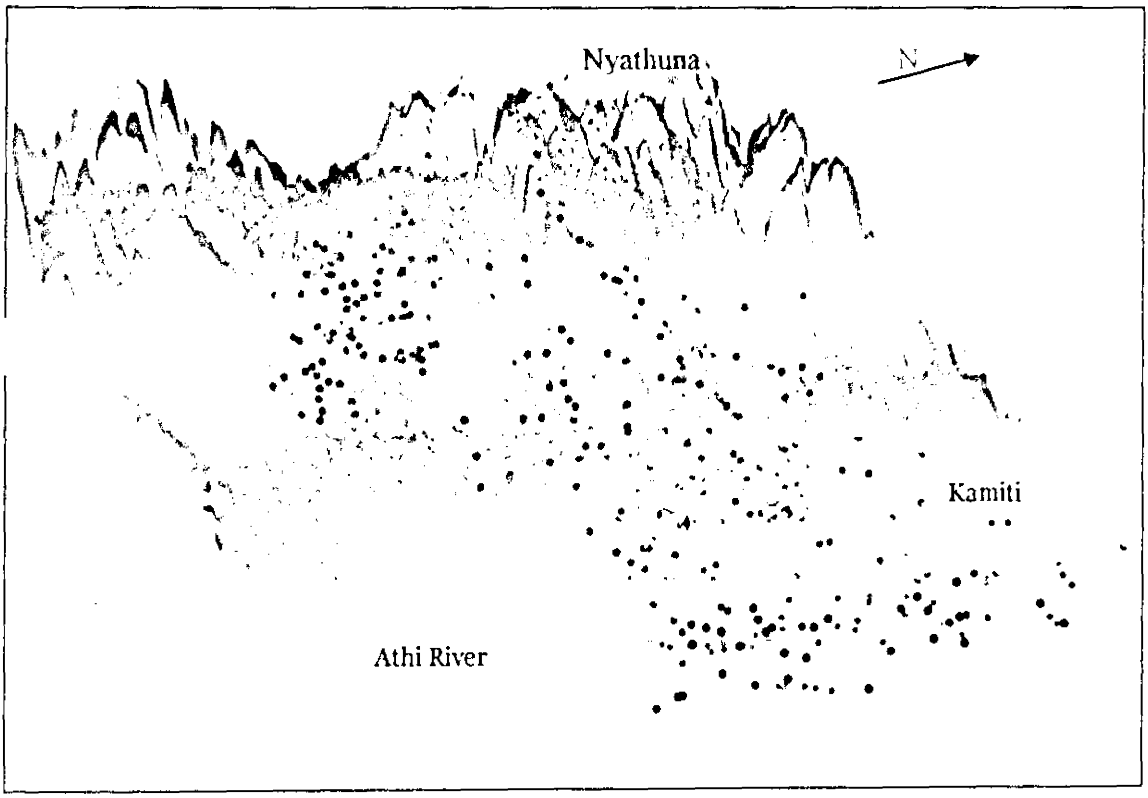

Figure 4.5: Rest levels in Nairobi boreholes superimposed on the DTM.

From the borehole distribution, it can be observed that most parts of the test area are represented in the analysis except the Ngong Hills to the south west and the Nairobi National Park to the south. Figure 4.6 depicts the position of sample boreholes on the geological map more clearly.

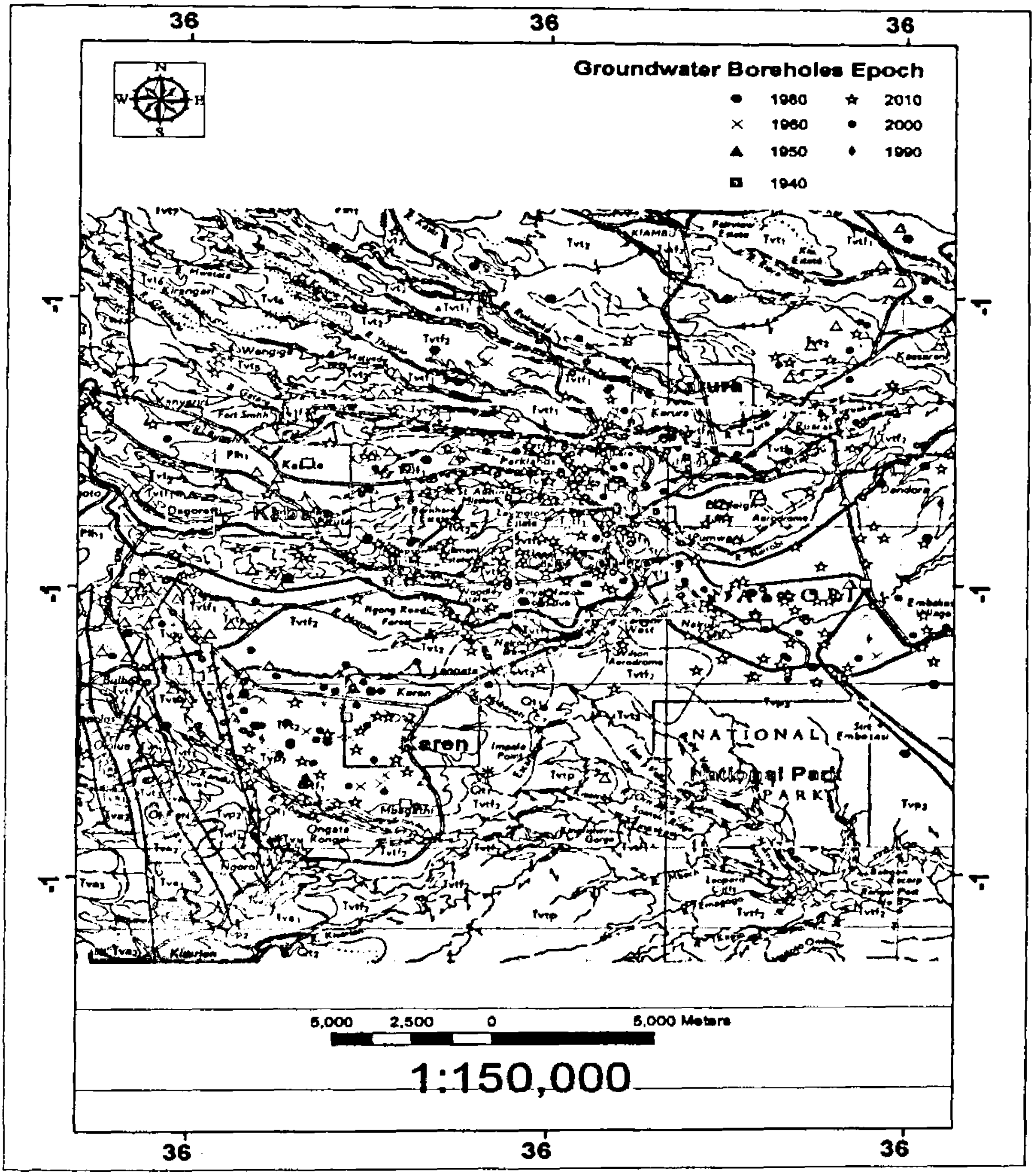

Figure 4.6: Position of sample boreholes on the geological map

Proximity to surface water sources sometimes influences rest levels in boreholes. Figure 4.7 presents a digitized drainage map that was used to study this relationship in the study area. The drainage pattern is dendritic and most rivers cut across all rock types without much meandering although several rapids can be found along the courses. Old rivers may have existed before the deposition of some of the geologic units as exemplified by existence of rounded conglomerates in materials recovered from different depths in various boreholes. The drainage map was overlaid on contour maps of the rest level surfaces and 3-D views generated for the various decades to enable identification of anomalies related to proximity to rivers.

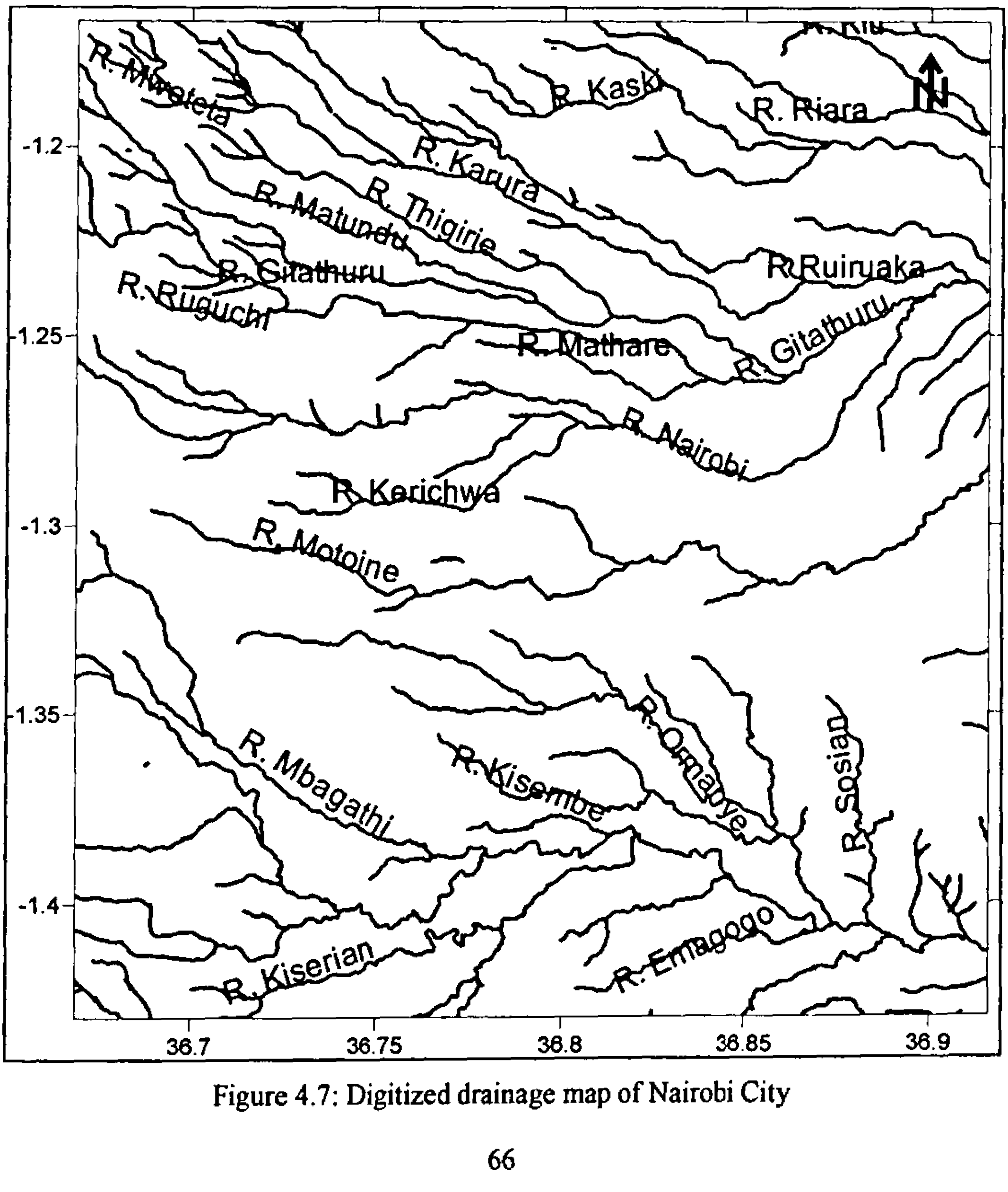

Figure 4.7: Digitized drainage map of Nairobi City

Figure 4.8 depicts a section of contour lines of groundwater surface produced by Kriging interpolation of rest level data for the decade that ended in 1990. On the figure is a superimposed drainage map (Figure 4.7), a buffer of 100 m along the rivers (blue), and, the borehole identities in that area (purple). River influence is observed in certain boreholes, such as boreholes 5018 and 4876 (highlighted), where groundwater rest levels are in the range of 30 to 60 m higher than those in the surrounding boreholes. Some local peaks that can be related to the structural geology are also observed. For instance, the south-western part of the area has several faults that act as recharge paths thus producing groundwater rest level anomalies. Borehole 4735 (shown) has a rest level that is almost 100 m higher than the levels in the surrounding boreholes.

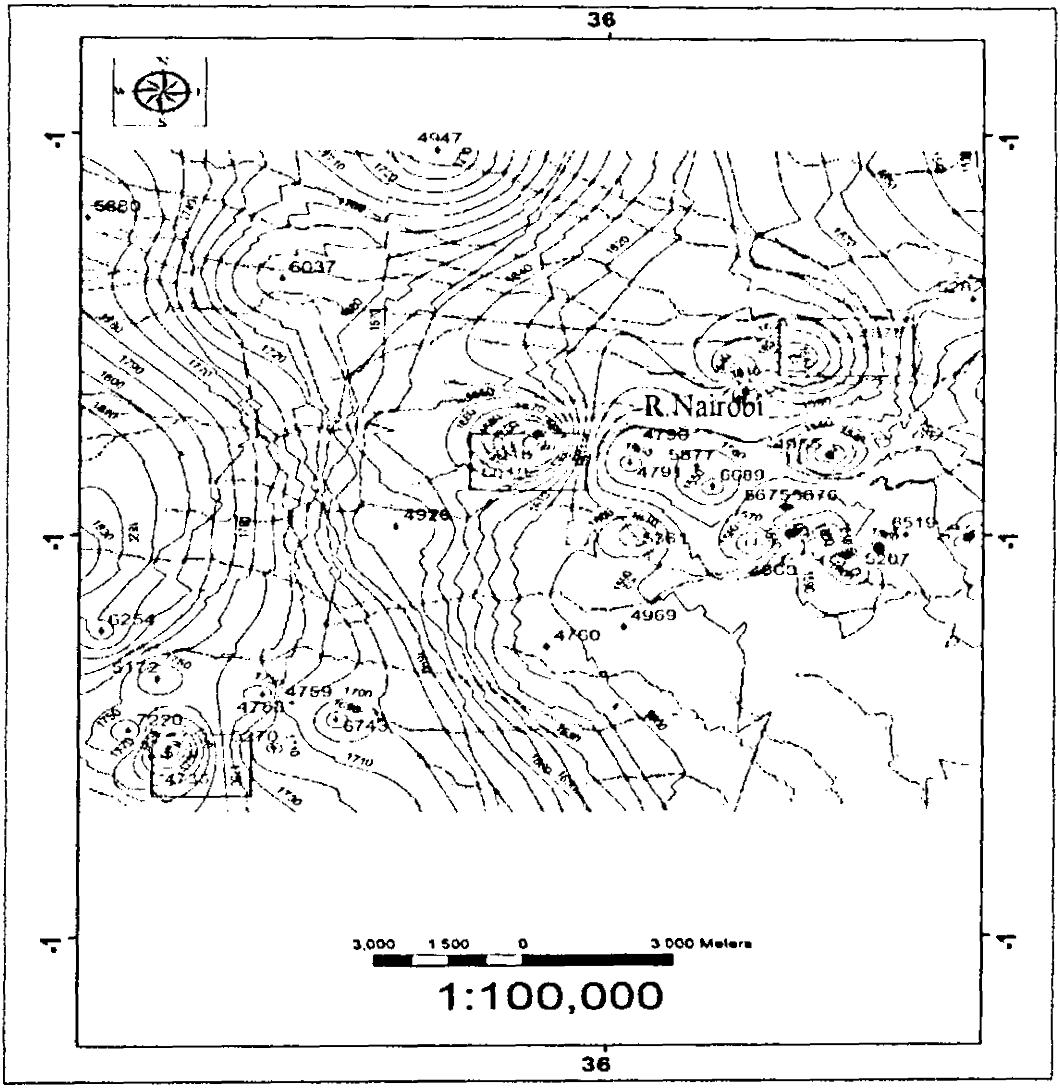

Figure 4.8: Superimposition of contours of rest level surface, rivers with 100 m buffer and borehole IDs for the decade ending in 1990.

Figure 4.9 depicts profiles of groundwater surfaces along a 20 km distance for decades ending in 1950 and 1990 compared with the terrain. The planimetric extent of this profile is also depicted in light blue in Figure 4.6. It is clear that there is a correspondence between the rest levels and the local terrain. The slope on the surface is reflected in the groundwater rest levels in the boreholes. For the decade ending in 1950, the rest level is closer to the terrain with an average altitude difference of 50 m; the 1990 profile shows a surface that is somewhat further from the terrain with an average altitude difference of 70 m, an average drop of 20 m from 1950. This trend is expected as it represents a 40 year period in which the number of boreholes increased from 400 to 1600.

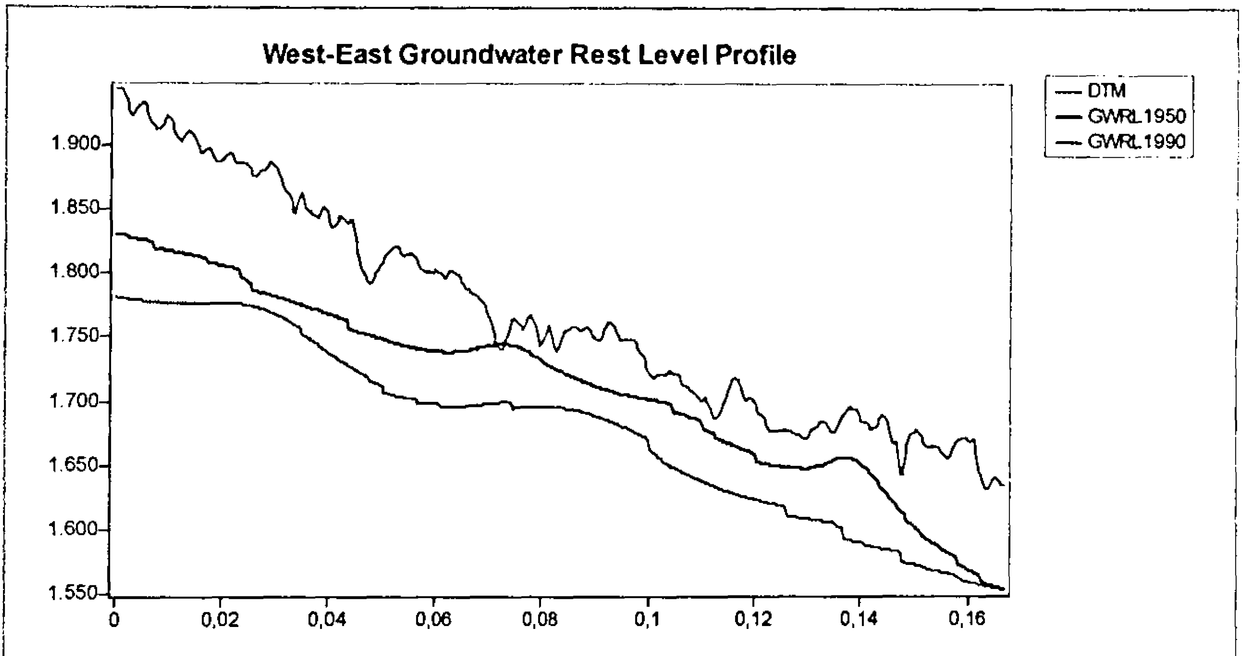

Figure 4.9: West-East cross section profile of rest level surfaces for 1950 and 1990 compared with local terrain (X axis in Decimal degrees; Y axis altitude in 10^{-3} metres).

To view the rest level variations in three dimensions, 3-D surfaces were generated for all decades. Figure 4.10 shows a superimposition of 1950 and 2010 3-D rest level surfaces together with rivers. It is clear that within the entire test area, the water rest level surface of 1950 (Pink) is above the one of 2010 (green). The exposed 2010 rest level surface to the north-western part of the area is a result of missing 1940s data.

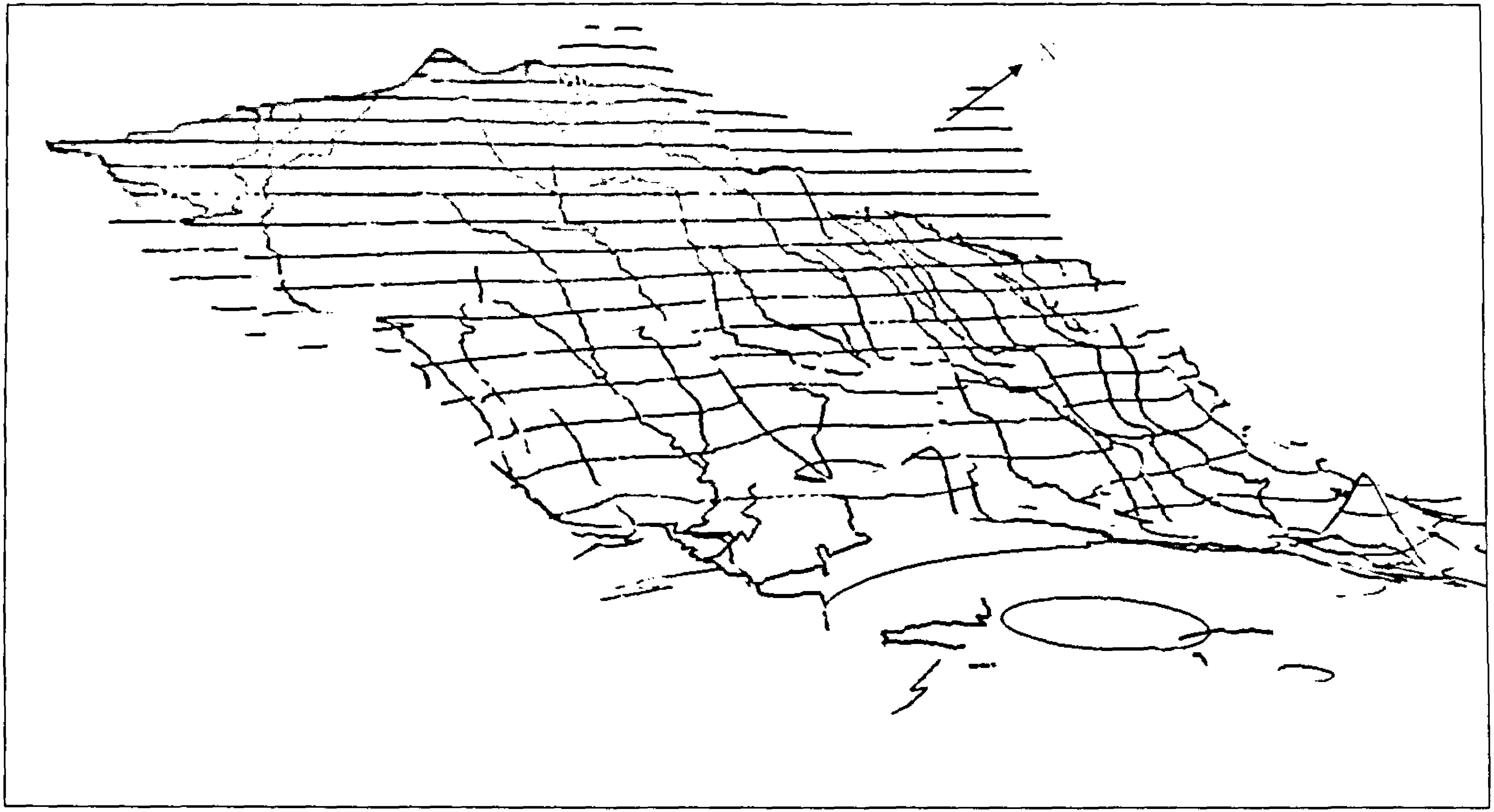

Figure 4.10: Overlays of 1950 (pink) and 2010 (green) rest level surfaces

Some anomalies that have no relationship with the structural geology or proximity to rivers were noted on the 3-D surfaces. For example, the two peaks from the 2010 surface that are visible in the middle of Figure 4.10 had to be explained. Rainfall hydrographs were used to relate the static levels with seasons. It was noted that high precipitation preceded drilling of the two boreholes. Figure 4.10 together with the analysis depicted in Figure 4.9, demonstrate that the rest levels have been descending over the past 70 years: from 1950 through 1990 and 2010. Figure 4.11 presents profiles representing rest level surfaces of 1950, 1990 and 2010 in comparison with the local terrain. The planimetric extent of this profile is depicted in purple in Figure 4.6.

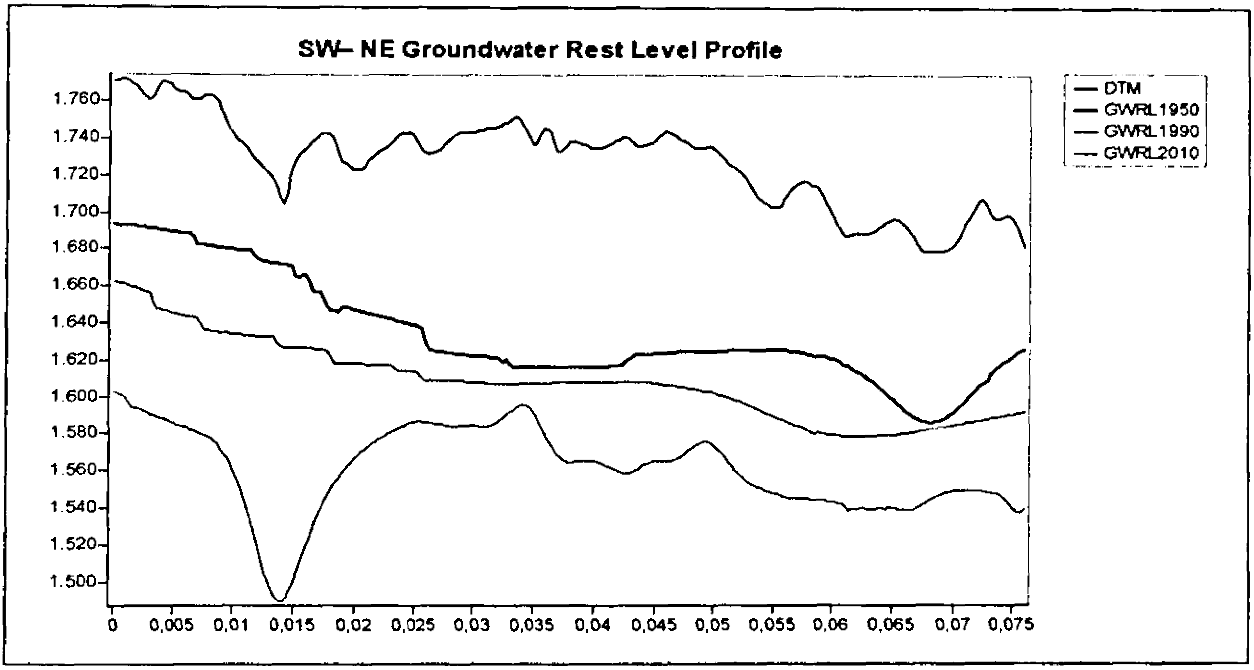

Figure 4.11: Profile of rest level surfaces at 1950, 1990 and 2010 compared with local terrain (X axis in Decimal degrees; Y axis altitude in 10^{-3} meters)

Statistics of the height difference between the 1950 and 2010 surfaces (after omitting the exposed area of the 2010 surface) in Figure 4.10 show that the majority of the area is experiencing a descent in water rest level, with a mean value of 61.5 m, and a standard deviation of 30 m. A few areas indicate otherwise, but these can be explained by local geological or hydrogeological effects as well as interpolation through areas without data. The cross sections in Figure 4.11 show a clear rest level drop and high correspondence between the rest level surface and the terrain for all three profiles along the cross-section.

4.1.3 Variation of water rest level as analysed on Surfer software

Rest level analyses on Surfer software were done with reference to the sea level because topography has great influence on the water levels in boreholes. To study the relationship between topography and rest levels in boreholes, contours were digitized as presented in Figure 4.12. The difference in elevation between the contours is 20 m. From the map, it can be observed that most of the contours run from north to south. The topography is thus gently rolling with the highest point standing at 2150 m above sea level on the north-western side and the lowest point at 1500 m above sea level on the south-eastern side.

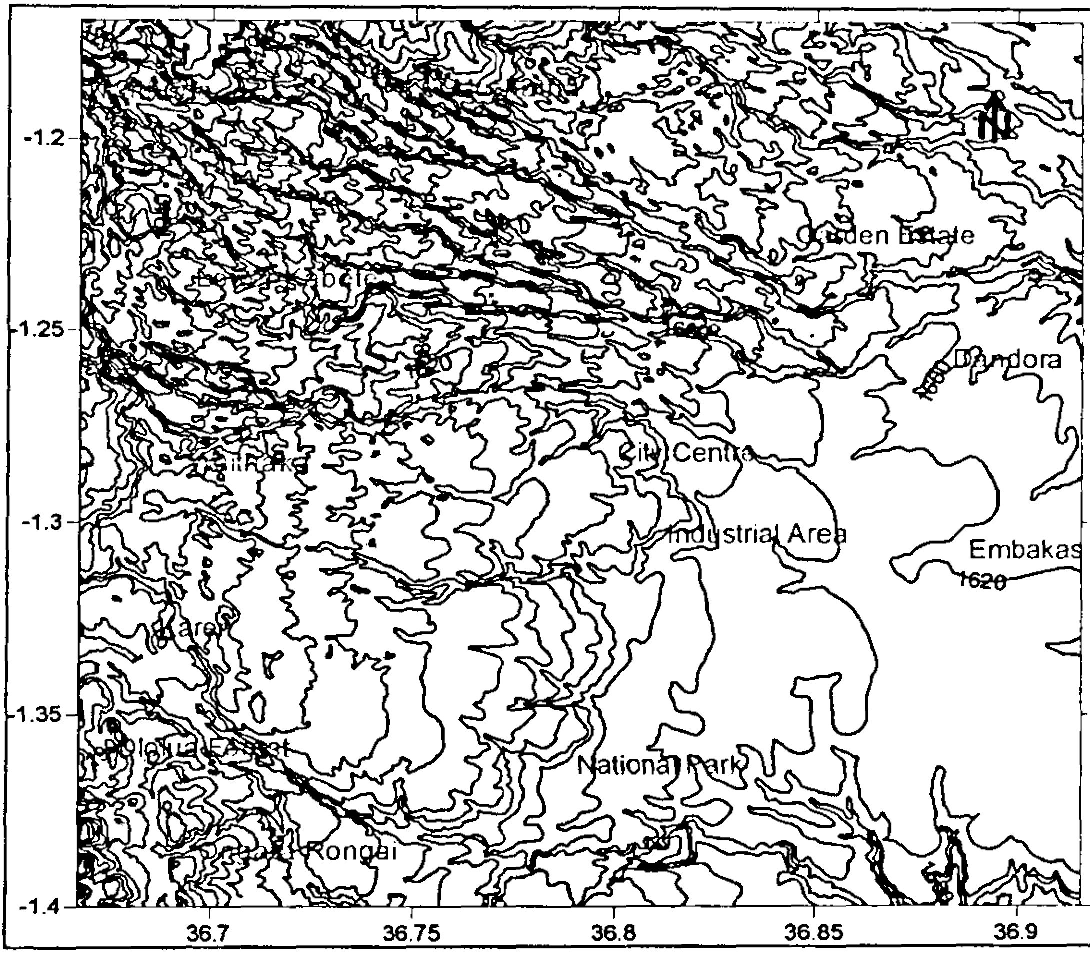

Figure 4.12: Digitized contour map of Nairobi City

The contour maps produced by analysis of spatial water rest level variations for the various decades on Surfer Software are presented in Figure 4.13 to Figure 4.19. The direction of ground water movement can be understood in the fact that ground water always flows in the direction of decreasing head. The rate of movement is dependent on the hydraulic gradient, which is the change in head per unit distance (Van Tonder *et al.*, 1998).

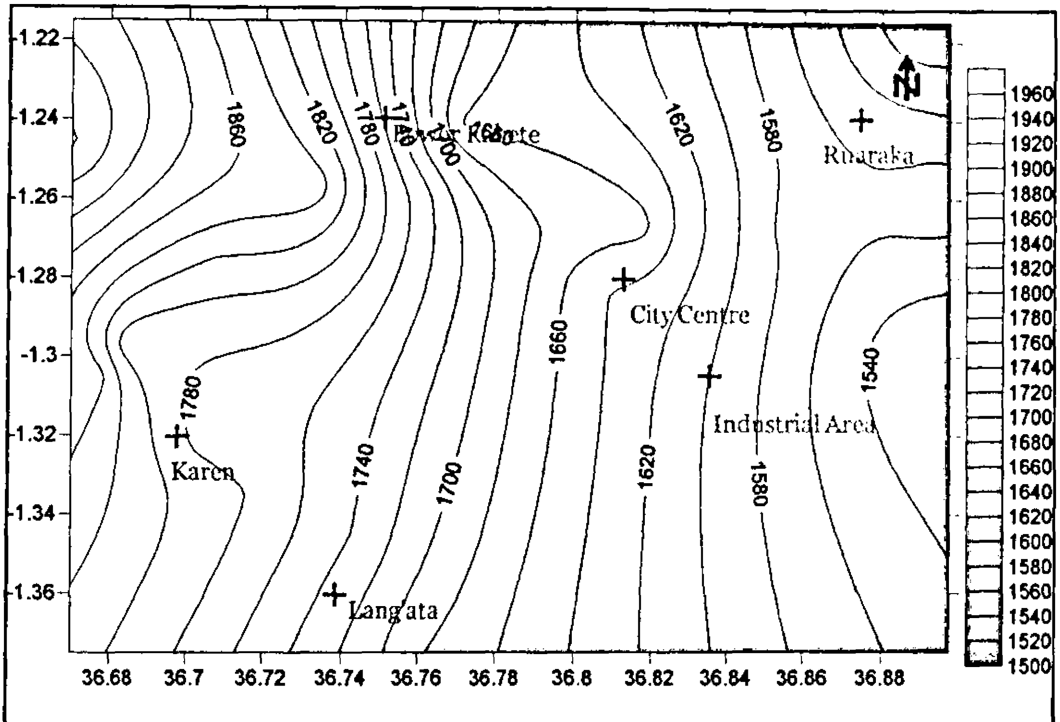

Figure 4.13: Contour map of water rest levels in boreholes drilled between 1920-1939

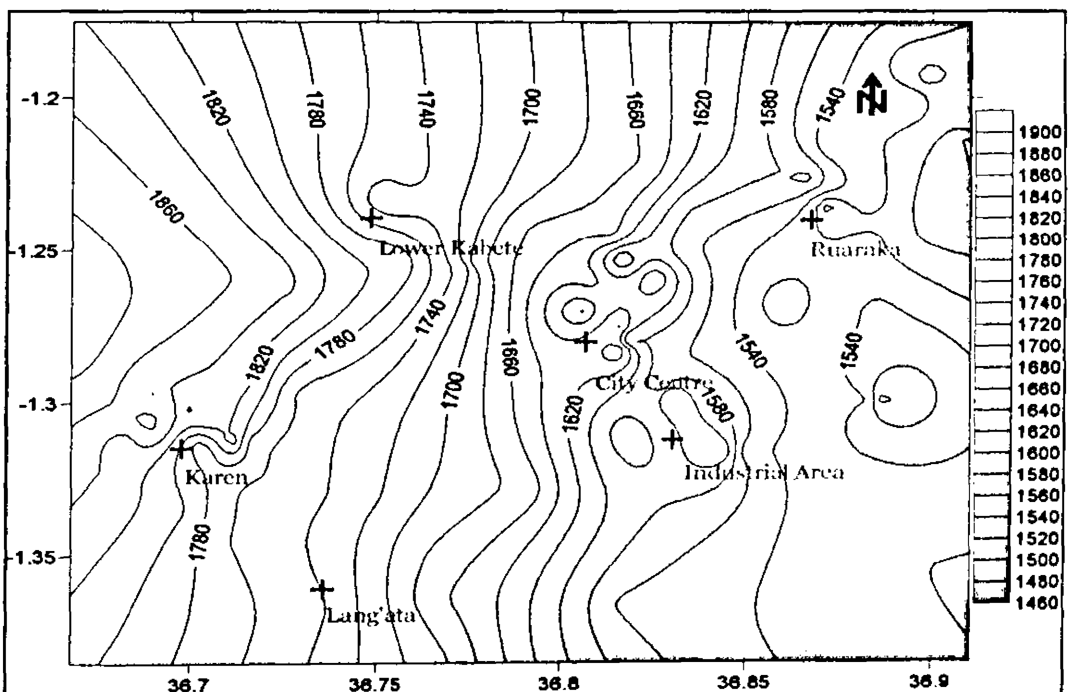

Figure 4.14: Contour map of water rest levels in boreholes drilled between 1940-1949

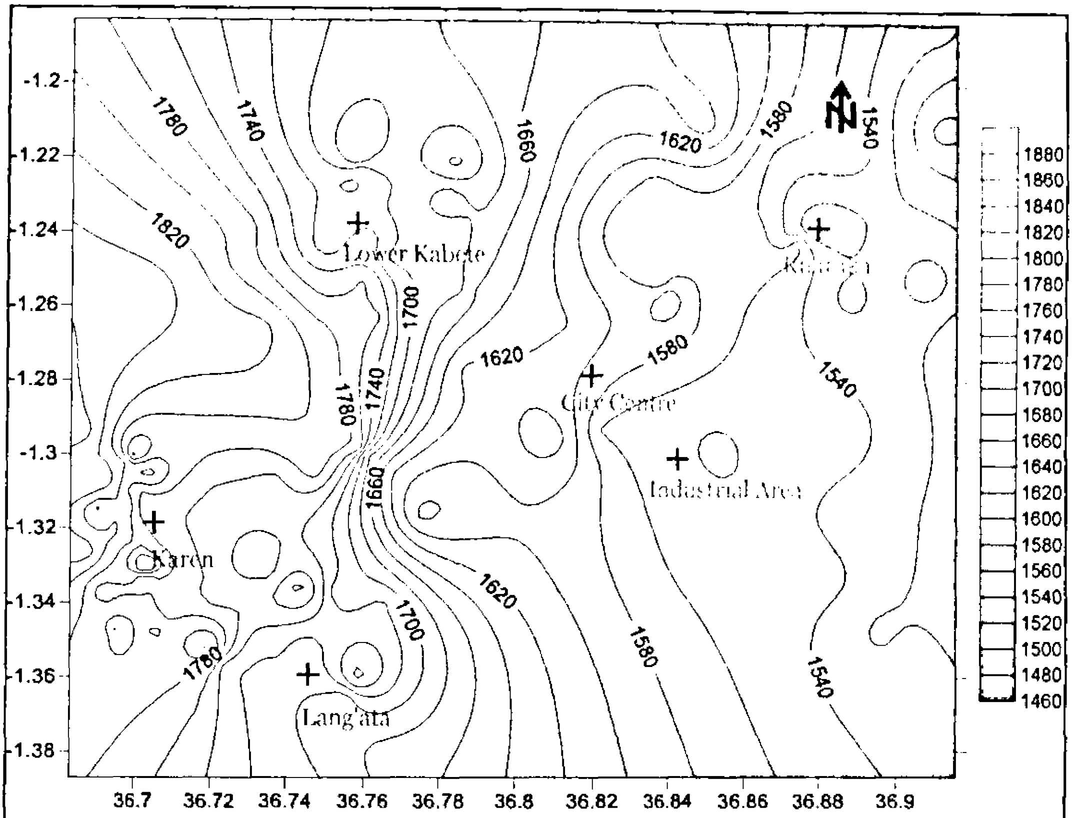

Figure 4.15: Contour map of water rest levels in boreholes drilled between 1950-1959

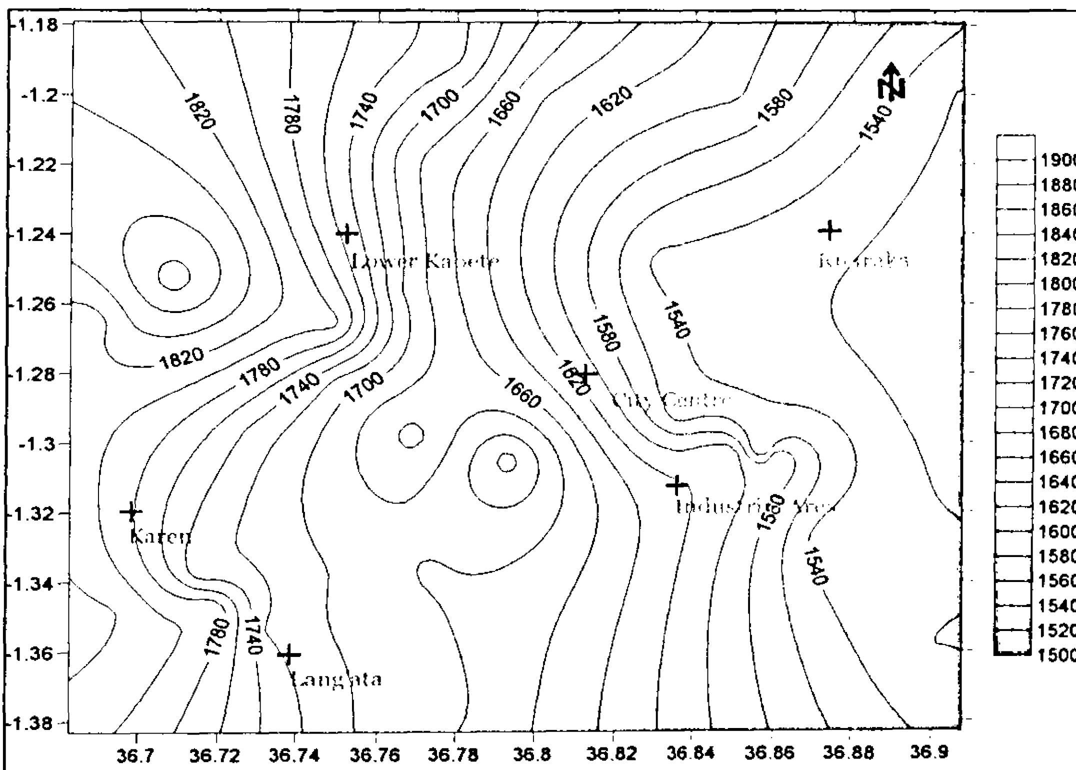

Figure 4.16: Contour map of water rest levels in boreholes drilled between 1960-1979

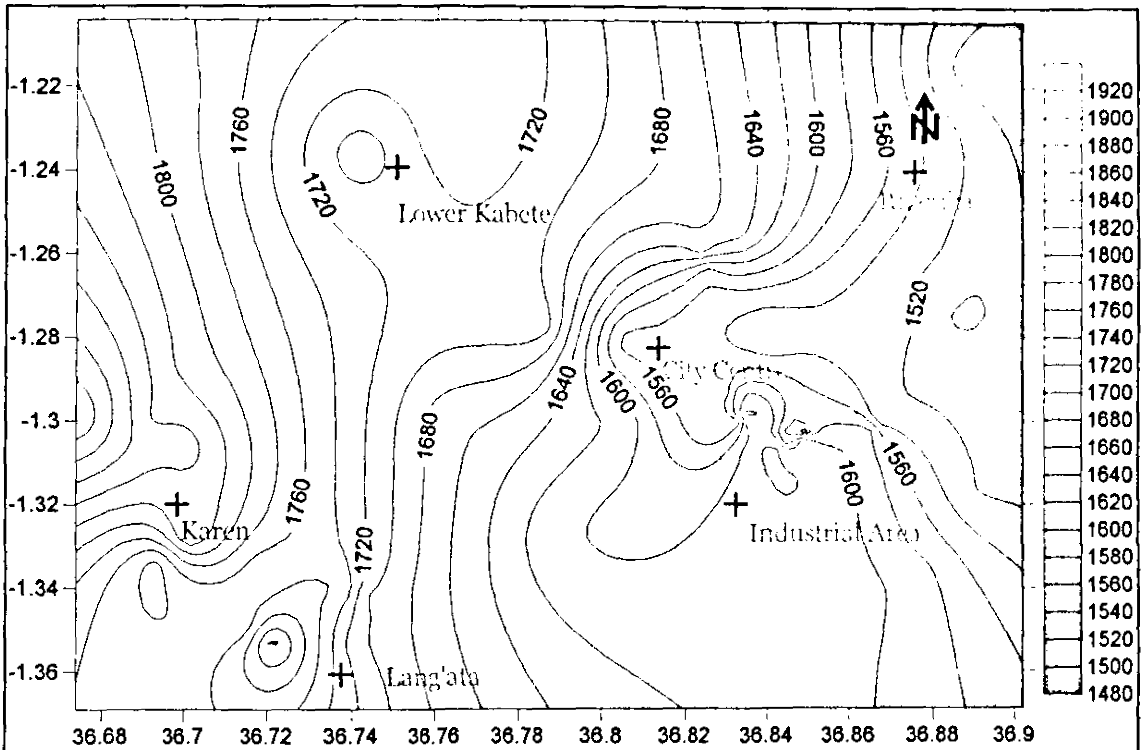

Figure 4.17: Contour map of water rest levels in boreholes drilled between 1980-1989

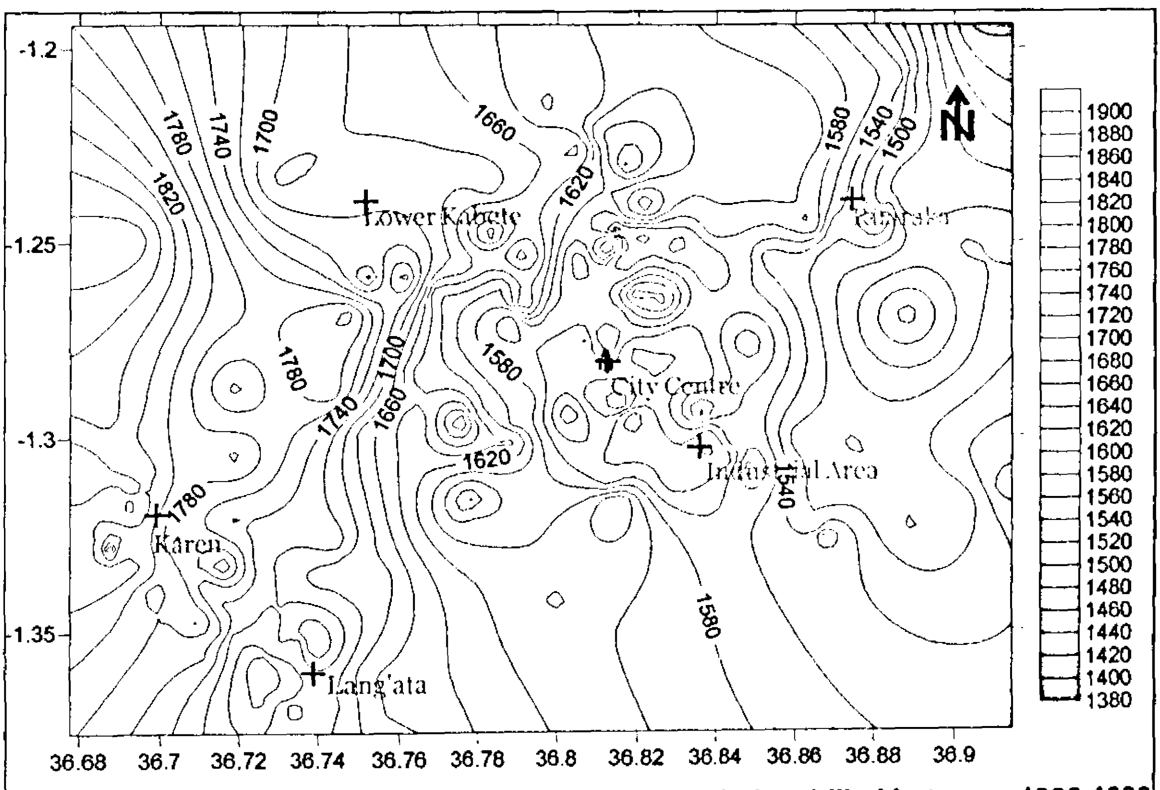

Figure 4.18: Contour map of water rest levels in boreholes drilled between 1990-1999

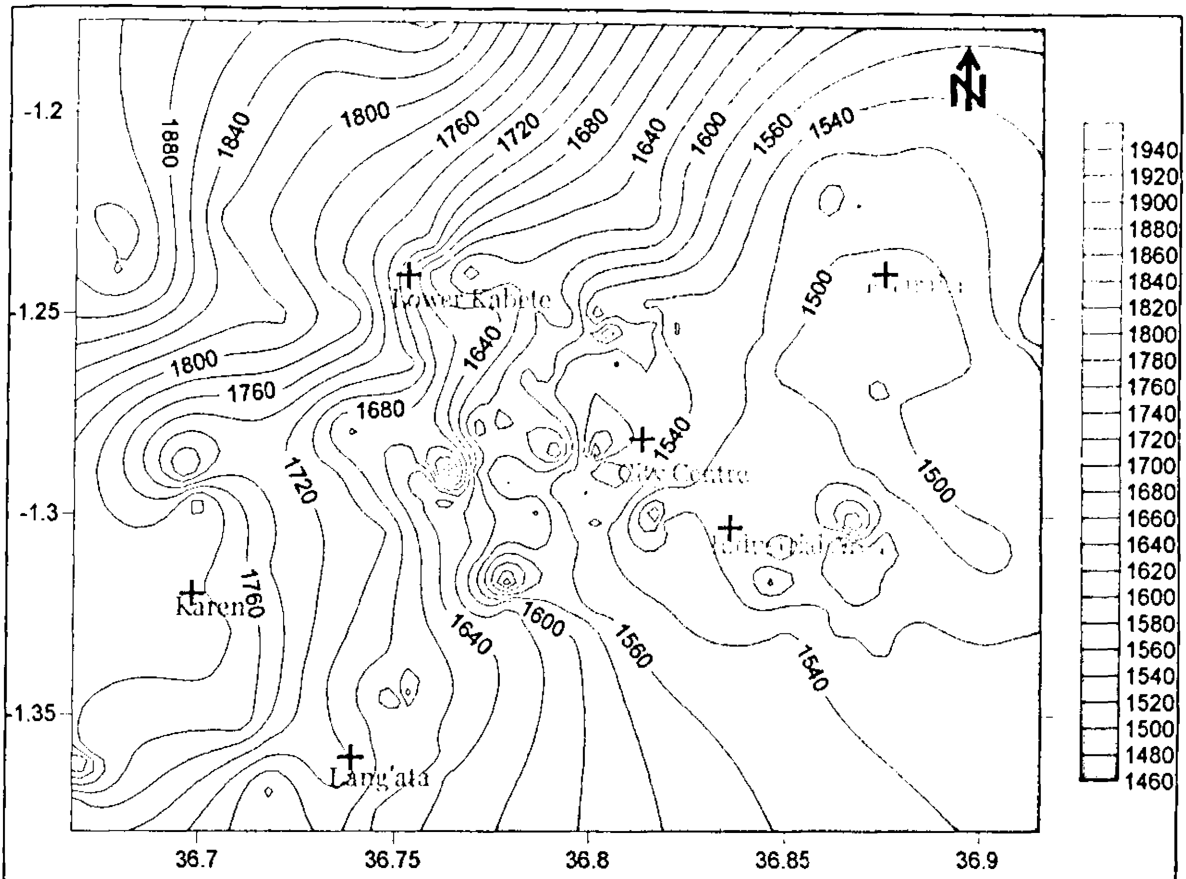

Figure 4.19: Contour map of water rest levels in boreholes drilled between 2000-2009

Surfer contours for 1927-1939 period show groundwater rest level surfaces with N-S trend in correspondence with the topographic contours. Change in contour trends started in the 1940s when several pockets developed in majority of the six main abstraction areas. The pockets evened out in the 1960-1979 decade. The recovery of rest levels in the 1960-1979 decade can be attributed to control on borehole drilling and restriction in abstraction (Mwangi, 2005). The control was accompanied by development of surface water supplies from Tana River Basin and regular measurement of rest levels in some selected boreholes. Owing to the curfew on drilling and the control in abstraction, water levels rose by an average of 40 m between 1960 and 1979 as expressed by the contours (Figure 4.15 and Figure 4.16). However after the control in drilling was lifted in the mid 1980, extensive abstraction of water commenced again leading to a fall of 100 m in the period between 1989 and 1999 (Figure 4.17 and Figure 4.18).

4.1.4 Subsurface profile in water supply boreholes

Most of the data used in analysing the subsurface profile was obtained by authors dedicated to water supply development. As might be expected, the database is variable due to differences in personnel and reporting procedures over time. Table 4.1 gives the stratigraphic profile and a summary of the thickness ranges of the main geologic units as established from water supply borehole logs. Most of the rocks are thought to have originated from Ngong´ volcano or through the faults in Limuru area.

Table 4.1: Geologic profile and summary of thickness ranges of geological units

Age	Geologic unit	Thickness range (m)	Total thickness range (m)
Quaternary	Alluvium, clays and swamp soils	1.5-22	50-430
Pleistocene	Limuru Quartz Trachytes	0-25	
Tertiary	Karura Trachytes	0-40	
	Kabete Trachytes	0-32	
	Kerichwa Valley Series	8-45	
	Nairobi Trachytes	0-91	
	Ngong Volcanics	0-58	
	Nairobi Phonolites	0-110	
	Kandizi Phonolites	0-60	
	Mbagathi Phonolitic Trachyte	0-100	
	Athi Series Tuffs and Lake Beds	16-305	
	Kapiti Phonolite	0-53	
Precambrian	Crystalline Rocks of Mozambique Belt	Extending beyond drilled depths	

In summary, thirteen geologic units are encountered in Nairobi with a variety of subsurface profiles. The depth of encounter of the geological units varies depending on the shape of the eroded surface at the time of deposition. The youngest rocks are exposed to the west and the oldest to the east. For comparison purposes, two profiles representing the extreme west and extreme east of the study area are presented in Figure 4.20.

Although the method of reporting aquifers differs from driller to driller, the average thicknesses were calculated from all the boreholes. From the core logs, it was found that the aquifer thicknesses are in the range of 28 m to 60 m. This corresponded well with the surfer contours (Figure 4.21) that show an average thickness of 40 m. The few localities with larger aquifer thicknesses are within areas with fracture traces as will be shown later. Depletion of aquifers in many parts of the world has lead to subsidence of the ground surface.

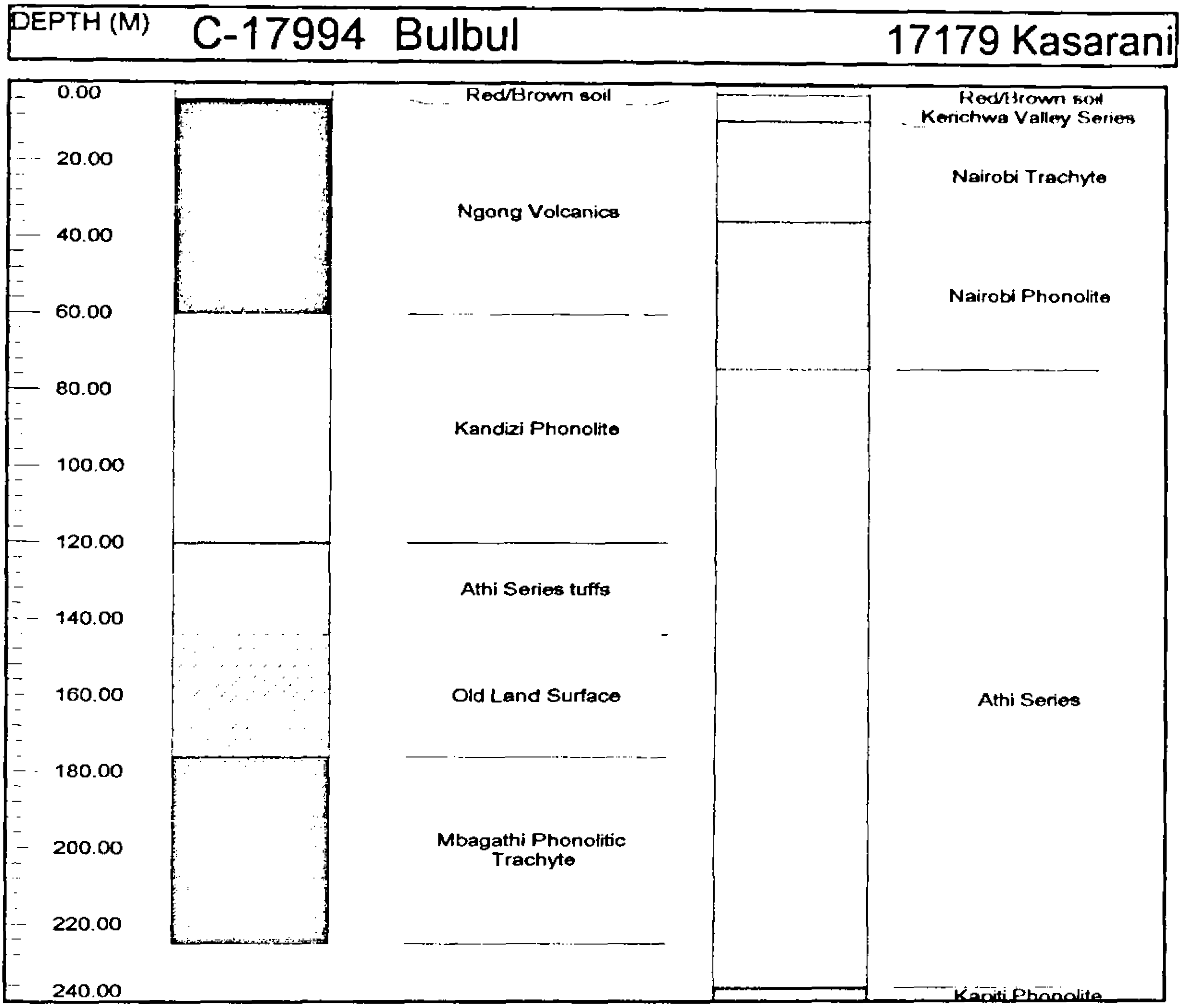

Figure 4.20: Subsurface profiles for the extreme east and extreme west of the study area

The following are depth ranges within which one would expect to encounter an aquifer: 18-30 m, 40-48 m, 76-134 m, 138-144 m, 150-162 m, 170-180 m, and 196-220 m. Up to six aquifers have been encountered in some boreholes. Main aquifers (yielding above 12 m^3/hr) are associated with volcanic sediments, basalts and faulted trachytes. The permeability of the volcanic sediments is relatively high and the sediments have provided typical extensive aquifers in Nairobi area (Mailu, 1987). The lateritic material underlying the near-surface soils and laterites on old land surfaces are often aquifers (Gevaerts, 1964).

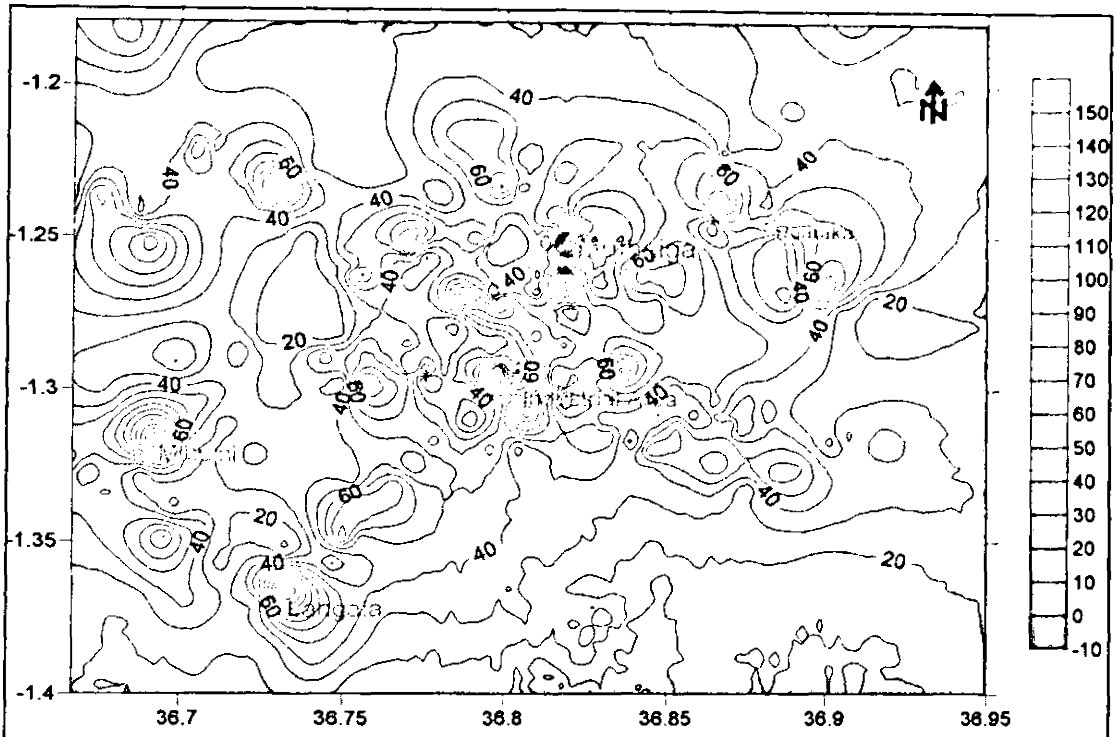

Figure 4.21: Surfer contours for aquifer thickness

4.1.5 Probable settlement due to water level lowering

An analysis of total and per annum change in rest levels for some observation boreholes is presented in Table 4.2. The most remarkable drops in rest levels are recorded at the following sites: Aga Khan Hospital with 126.2 m drop and an average annual drop of 5.0 m; Uchumi Ngong' Road with 87 m drop and an average annual drop of 7.91 m; Unilever Industrial Area with a total drop of 87.5 m and an annual drop of 2.65 m; St Laurence University along Miotoni Road in Karen with total drop of 71 m and annual drop of 8.88 m. A few boreholes such as the one at the Hindu temple in Parklands and Kenya Polytechnic Men's Hostels show a rise in rest level resulting from either shut-down or controlled pumping. The drop at St Lawrence University borehole occurred during test pumping.

Table 4.2: Total and per annum change in water rest levels in monitoring wells

BH No.	Owner	Date drilled	Total depth (m)	Initial WRL (m)	Change in WRL (m)	Date measured	Operating years	Average p.a change (m)
12357	Hindu Temple Pl	1999	204	160	29.80	2008,March	9	3.31
168	NCC Kabete	1941	128	13	-2.50	2008,March	Not pumping	
14539	Riverside Park	2004	280	149	-17.30	2008,March	4	-4.33
10883	Hill Crest Karen	1994	300	110	-14.00	2008,March	14	-1.00
10333	Hurlingham	1993	250	152	-31.00	2008,March	15	-2.07
3958	Karen Club	1973	312	114	-8.00	2008,March	Not pumping	
13860	Hotel Boulevard	2003	255	115	-2.90	2008,March	5	-0.58
6310	KEWI South C	1985	169	77	-54.20	2007 March	22	-2.46
8697	Trufoods	1989	164	105	-27.90	2008,March	19	-1.47
4790	STATE HOUSE	1980	200	146	-19.20	2007 March	27	-0.71
4147	Unilever	1975	258	79	-87.50	2008,March	33	-2.65
11701	Public Service Cl	1997	250	166	-4.10	2007 March	Not pumping	
11592	Uchumi Hyper	1997	250	143	-87.00	2008,March	11	-7.91
5050	Aga Khan Hosp	1982	171	21	-126.20	2007 March	25	-5.05
14559	Kabansora Millers	2005	200	102	-4.50	2007 March	2	-2.25
12885	Kenya Poly Host	2000	200	112	-31.00	2008,March	8	-3.88
15129	Jorgen L Karen	2006	170	92.7	-11.00	2008,March	5	-2.20
13594	KICC	2002	250	105.4	-31.30	2008,March	6	-5.22
13069	St Laurence Univ.	2000	320	55	-71.00	2008,March	8	-8.88

To calculate the settlement that might result from groundwater drawdown, rest levels taken during monitoring and test pumping were used. From the monitoring data, the actual water level measurements show that the minimum annual water level drop is 0.58 m while the average drop is 2.2 m. If the rest level measurements are extrapolated to 1930, the minimum water level drop would be expected to be 46.4 m after 80 years of active abstraction.

The cumulative drop in rest level calculated from the records of initial rest levels measured before test pumping is 79.34 m. GIS depicts an average drop of 62 m in the period between 1940 and 2010; an average drop of 0.89 m/year or 8.9 m per decade. So from 1927 to 1939, the water level drop could be 11.5 m and the total drop would then be 73 m. Considering the lowest expected rest level drop of 46.4 m from actual measurements and the calculated cumulative average drop of 79.34 m, the average drop of 73 m determined on the GIS models was taken as representative for calculating settlement in areas without measurements. In areas with actual water level measurements, drops of greater than 73 m were used for calculation of settlement.

Presented here is an example of settlement calculation for Ruaraka area where 17 wells were considered. The upper confining layer is Nairobi phonolite and the confined aquifers are within the 142 m thick Athi Series. The average aquifer thickness is 48.7 m while the aquitards are 38 m thick. The hydraulic separators are 55.3 m thick. In 1963, borehole C-944 at East African Breweries showed a 33 m depression of pressure surface after 14 years in operation, an average drop of 2.357 m per year. If this trend could be assumed to continue in the successive years, after 48 years the drop would be assumed to be 33+ (2.357*48) = 146 m, so an assumption of 73 m drop is very modest. Settlement values calculated for Ruaraka and 22 other localities are as shown in Table 4.3.

Table 4.3: Calculation of probable settlement due to groundwater withdrawal

Locality	Aquifer (m)	Aquitard (m)	Δh (m)	β_w	Ss	S	Δp	Δm (m)	ΔM (m)	$\delta_{T\,(m)}$
Parklands	48.35	52	126.2	4.60E-10	0.000221	0.0107	474.3	5.17E-02	1.452279	1.5E+00
Westlands	40.17	54	73	4.60E-10	0.000205	0.0083	394.1	3.31E-02	0.809597	8.4E-01
Chiromo/R-Side/ Kileleshwa	46.92	60	73	4.60E-10	0.00117	0.0549	460.3	2.58E-01	5.124936	5.4E+00
Muthaiga	40.64	56	73	4.60E-10	0.000637	0.0259	398.7	1.05E-01	2.605295	2.7E+00
Langata	57.27	64	100	4.60E-10	0.000384	0.0220	561.8	1.26E-01	2.457369	2.6E+00
Dam Est/Kibera/Woodley	48.71	32	87	4.60E-10	0.000111	0.0054	477.8	2.64E-02	0.309492	3.4E-01
Mombasa Rd	38.00	70	73	4.60E-10	0.00033	0.0126	372.8	4.77E-02	1.687645	1.7E+00
Industrial Area	48.70	36	87.5	4.60E-10	0.000679	0.0331	477.7	1.61E-01	2.139388	2.3E+00
Embakasi	38.50	54	73	4.60E-10	0.000326	0.0126	377.7	4.83E-02	1.28499	1.3E+00
Runda/Kitisuru	52.60	46	73	4.60E-10	0.000174	0.0091	516.0	4.81E-02	0.583394	6.3E-01
Riruta/kangemi/Kawangw	45.75	74	73	4.60E-10	0.00014	0.0064	448.8	2.92E-02	0.754509	7.8E-01
Gigiri/Rosslyn	47.00	46	73	4.60E-10	0.000213	0.0100	461.1	4.70E-02	0.714468	7.6E-01
Karen	38.38	52	77	4.60E-10	0.000573	0.0220	376.5	8.44E-02	2.29407	2.4E+00
Lowe Hill/N-West	58.40	78	73	4.60E-10	0.000207	0.0121	572.9	7.07E-02	1.17975	1.3E+00
Lavington Kilimani/Thomp	43.81	70	73	4.60E-10	0.000112	0.0049	429.8	2.15E-02	0.571536	5.9E-01
Upper Hill/Milimani	58.30	64	73	4.60E-10	0.000942	0.0549	571.9	3.20E-01	4.399533	4.7E+00
Lower kabete/Wangige	55.18	38	73	4.60E-10	0.000166	0.0091	541.3	5.04E-02	0.459401	5.1E-01
Ngara/Pumwani/Eastleigh	50.20	58	73	4.60E-10	0.00012	0.0060	492.5	3.02E-02	0.507743	5.4E-01
Ongata R/ Ngong'/Kiserian	34.82	32	73	4.60E-10	0.002397	0.0835	341.6	2.91E-01	5.59996	5.9E+00
Buru/Umoja/Kayole	39.70	36	73	4.60E-10	0.000322	0.0128	389.5	5.07E-02	0.845461	9.0E-01
Kikuyu/Dagoretti/Kinoo	32.37	48	73	4.60E-10	0.001291	0.0418	317.5	1.35E-01	4.524782	4.7E+00
Ruaraka	48.70	38	73	4.60E-10	0.000875	0.0426	477.7	2.08E-01	2.428328	2.6E+00
National Park	51.00	65	100	4.60E-10	0.000498	0.0254	500.3	1.30E-01	3.237255	3.4E+00
Calculated average	**46.24**	**53.17**	**79.07**	**0.00**	**0.00**	**0.02**	**453.59**	**0.10**	**2.00**	**2.10**

S_s = the specific storage S = Storativity (dimensionless)

β_w = is the compressibility of the water (4.6×10^{-10} m²/N).

Δm = layer thickness change ΔP= Fluid pressure change

δ_T = Total settlement

4.1.6 Fracture traces

Several fractures were traced in the Vicinity of Nairobi City as shown on Figure 3.1.The traces indicate that the City Centre subsurface may have been affected by two sets of faulting: NW-SE and E-W. One NW-SE fracture trace at the City Centre affects the Nairobi Trachyte that is prominent at Upper Hill area but is not encountered beyond Uhuru Highway. A borehole at the Latter-Day Saints church at Lower Hill encountered a small thickness of the Nairobi Trachyte but BH C-1987 at Uhuru Highway railway crossing did not encounter it, showing that it suddenly ends within the Railway Golf Club.

The second City Centre fracture trace is observed at Uhuru Highway railway crossing through Moi Avenue line to the Premier Academy borehole at Parklands that has exceptionally high yield like BH C-1987 at railway crossing. The third fracture trace is mapped in several boreholes in Industrial area and emerges at the Railway Station and can be observed as far north as Muthaiga where distress in structures is reported. Given the three faults cutting through City Centre in the N-S direction, the parts covered with deep soft soils seem to represent a graben while those covered by KVS represent the horst.

The E-W faulting is represented by Kenyatta Avenue-Valley Road where there are different subsoil types on either side of the road. The other important fracture traced is that along Ngong' Road where deep soft soils are encountered in construction excavations. aquifers and rest levels are very deep along Ngong Road. Engineering consultants for the NSSF building at Upper Hill also reported encounter of sudden voids within very competent Nairobi Trachyte.

4.1.7 Electrical resistivity survey

Results for the resistivity survey are presented as curves for various sounding sites plotted on both Interpex and Earth Imager softwares. Each horizontal flat surface on the models represents a geoelectric layer whose depth and thickness can be obtained from the curve. Figure 4.22 shows Interpex and Earth Imager curves for Uhuru Park site; Interpex identified five layers whereas Earth Imager identified 9 layers as shown on Table 4.4. The other curves (Figure 4.23 to 4.28) were interpreted in the same manner. The difference between the models in Interpex and Earth

Imager software are due to the number of iterations that can be performed during modelling. It is possible to manually shift points to fit the curve in the Earth Imager not in the Interpex.

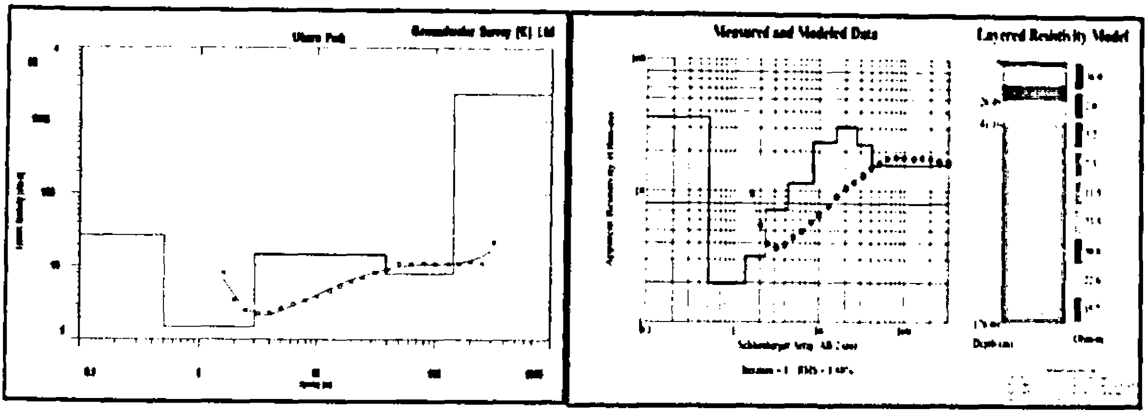

Figure 4.22: Sounding curves and layered model for Uhuru Park (VS-34)

Table 4.4: Layer parameters identified on Interpex and Earth Imager curves for VS-34

Interpex Curve			**Earth Imager Curve**		
Layer	Resistivity (ohm-m)	Thickness (m)	Layer	Resistivity (ohm-m)	Thickness (m)
1	27	0.5	1	36	0.5
2	1.8	2.5	2	2	0.7
3	15	37.5	3	3.2	0.9
4	8	120	4	7.3	2.0
5	2400	∞	5	11.5	3.1
			6	23.5	6.7
			7	30.8	11.8
			8	22.6	15.5
			9	15.7	∞

∞- means extending beyond the modelled depth

Curve marching involves comparison of VES curves obtained in a resistivity survey with Master Curves obtained from several experimental studies. The comparison is based on the values of apparent resistivity of the subsurface layers detected in a resistivity curve. The Master Curves include: H type where $\rho_1 > \rho_2 < \rho_3$; K type where $\rho_1 < \rho_2 > \rho_3$; A type where $\rho_1 < \rho_2 < \rho_3$; and, Q type where $\rho_1 > \rho_2 > \rho_3$. On marching the curves for various VES sites, it was observed that where the subsurface profile is similar, the resistivity curves have a similar shape. Seven shapes of curves were obtained for the entire study area. A summary of the geoelectric parameters (resistivity, ρ_i and thickness, h_i) obtained from the sounding curves analysed on Interpex 1-D sounding software as well as the aquifers identified are as shown in Appendix 2.

VES at Uhuru Park site (Fig. 4.22) recorded low resistivity values ranging from 4.3 to 36.0 Ohm-m indicating the dominance of clays and weathered materials. This corresponds well with the core log recovered from the water supply borehole at Serena Hotel (C-9754) that indicates the presence of alternating layers of weathered and fresh materials. Seventeen sites with curves of similar shape as the VS-34 (HAKQ type) are indicated in the same colour on Figure 3.1. The stratigraphic column for Serena Hotel site is presented later in section 4.1.8

The second shape of curve obtained is KHQ type represented by the Interpex curve for University of Nairobi Sports Field (Fig 4.23). This curve shows a low resistivity layer that extends from the surface to 21.5 m depth, while Earth Imager curve shows six low resistivity layers extending to 13.9 m. The low resistivity near-surface layers correlate well with a geotechnical borehole log at Museum Hill Bridge that shows layers of clay, silt and some peat extending to 22 m. At 250-320 m depth, resistivity values dropped from a high of 85 ohm-m to a minimum value of 15 ohm-m which was indicative of an aquifer underneath. The core log from Boulevard Hotel indicates that first aquifer was encountered at 150-162 m and the second aquifer at 204-323 m, correlating well with the resistivity values. The stratigraphic column for the University of Nairobi borehole is presented later in section 4.1.8.

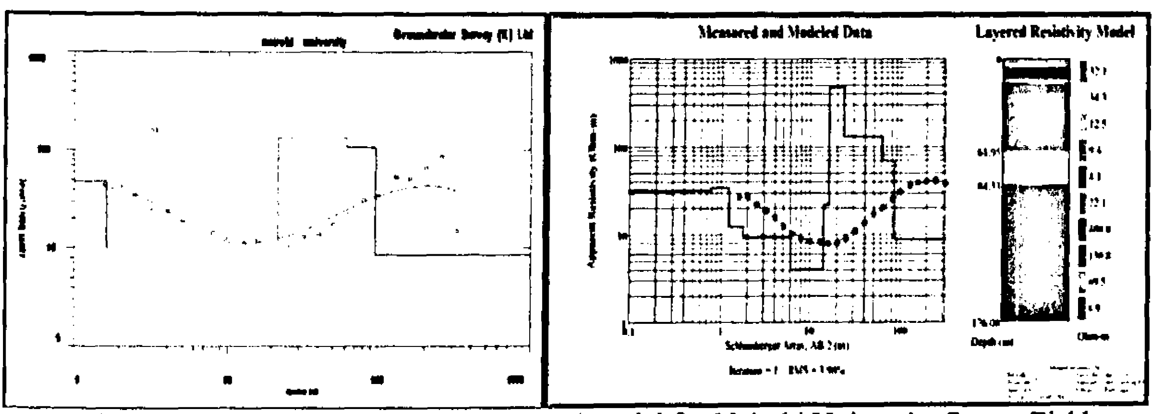

Figure 4.23: Sounding curves and layered model for Nairobi University Sports Field (VS-35)

The third shape of curve obtained is the AHKA type represented by Figure 4.24. Twelve sites with curves of similar shape as 4.24 are indicated in the same colour on Figure 3.1. The fourth type of curve obtained is AKQ type as represented by Figure 4.25. Eight sites with curves of similar shape as 4.25 are indicated in the same colour on Figure 3.1, most of them in east Nairobi and Karen- Langata.

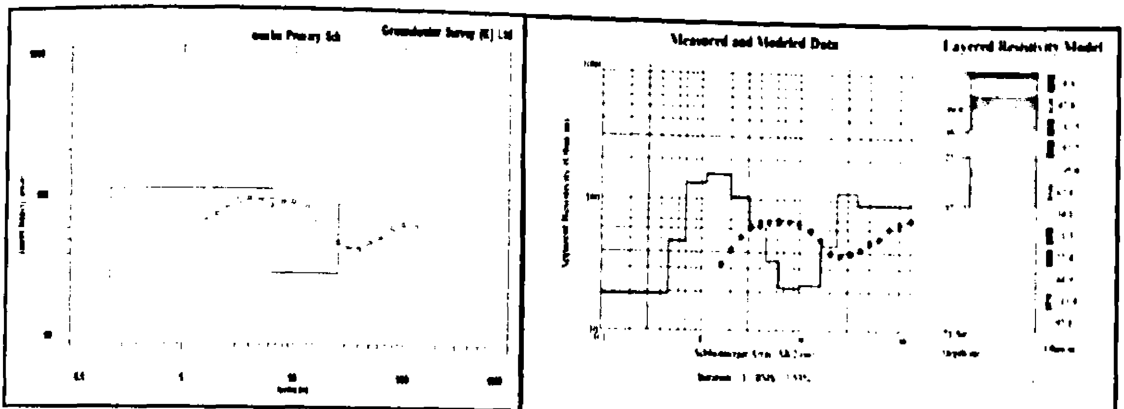

Figure 4.24: Sounding curves and layered model for Nembu Primary School (VS-16)

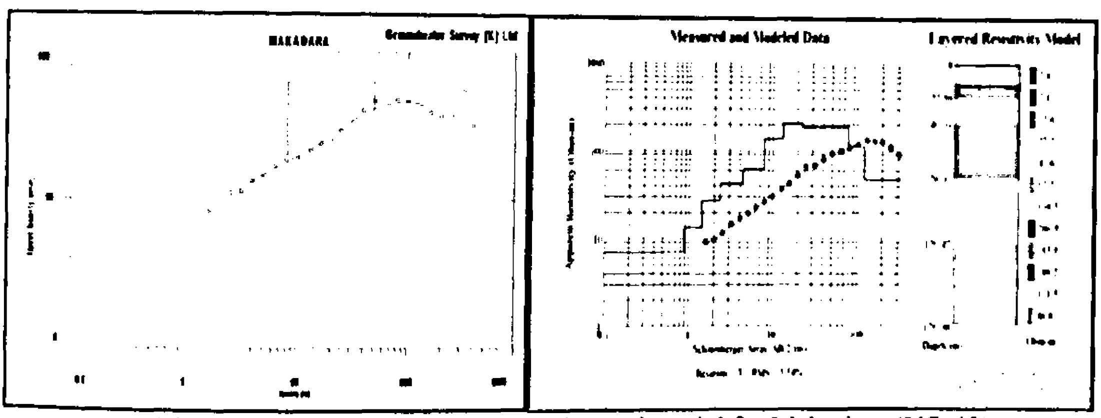

Figure 4.25: Sounding curves and layered model for Makadara (VS-43)

The fifth shape of curve is AHKQ type while the sixth shape of curve is represented by Figure 4.27. VES was carried out at Bulbul (Figure 4.27 and Figure 4.28), one line running along the fault and the other line running across the fault. The resistivity variation along the fault forms a curve of HQA type while across the fault, the curve is HAQK type. The near surface layers along the fault have the resistivity values below 5 ohm-m up to 5 m depth and less than 30 ohm-m up to 30 m depth. Across the fault, the ground surface was very dry, with resistivity value of 210 ohm-m at the ground surface gradually reducing to 46 ohm-m at 4 m depth after which it remained below 65 ohm-m. Four sites with HQA type curves and two sites with HAQK curves are indicated in the same colours in Figure 3.1.

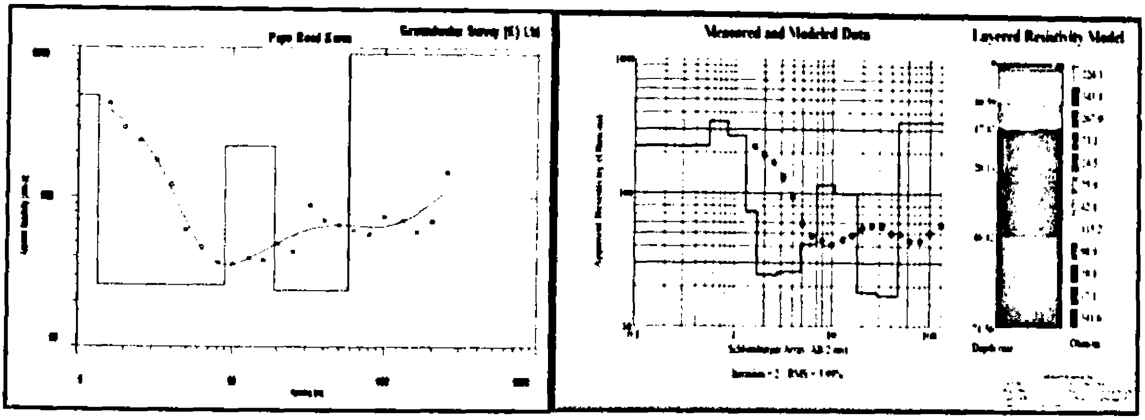

Figure 4.26: Sounding curves and layered model for Pepo Road (VS-6)

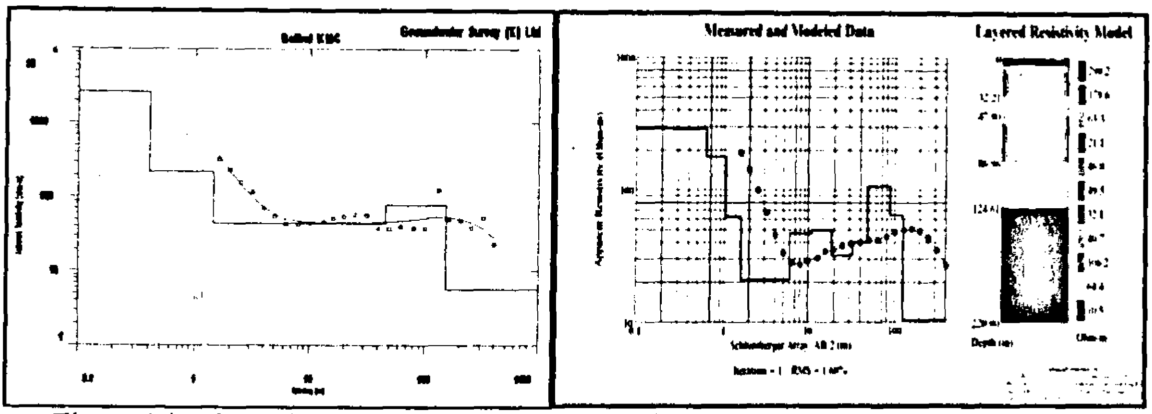

Figure 4.27: Sounding curves and layered model for Bulbul KMC across faultline (VS-1)

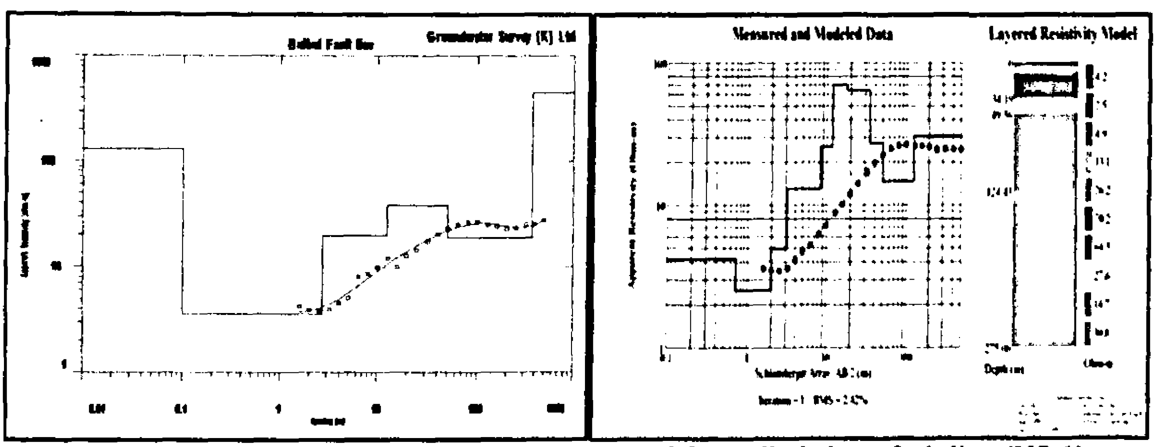

Figure 4.28: Sounding curves and layered model for Bulbul along fault line (VS-1)

If the two layered resistivity models for Bulbul are compared at 49.56 m and 47.8 m, they have resistivities of 14.7 ohm-m and 106.2 ohm-m respectively. This indicates that the subsurface varies in the two directions owing to the presence of the fault, which a casual look on the ground could conclude that it is a dry river valley. But the two Bulbul curves agree at depths below 124 m where there is a low resistivity layer indicating saturation. A nearby borehole drilled during the period of this study (C-17994 in Figure 4.20) encountered Athi tuffs and old land surface sediments at 120-178 m. The major aquifer was encountered within the old land surface sediments. The overall subsurface profile in Figure 4.20 agrees more with the resistivity model across the fault where the Athi tuffs and old land surface appear as one geoelectric layer.

4.1.8 Predicting mechanical properties from resistivity data

Unconfined Compressive strength, ultimate and allowable bearing capacity as well as punching shear strength were related to the resistivity values according to Equation 3.14 and 3.15 (Ramamurthy, 2008). Results for five vertical electrical sounding sites are presented in Appendix 7. The bearing capacity values calculated using Equation 3.12 are almost ten times more than the tested values. Also, the difference in bearing capacity between competent and weak layers is small. This is so because the apparent resistivity values obtained are actually an average along the path travelled by the current. The punching shear strength (τ_p) values obtained by Equation 3.14 are not useful because they are all negative. Positive values are obtained when the resistivity is greater than 3400 ohm-m.

A comparison of borehole logs and resistivity models for four of the VES sites is presented in Figure 4.29 to 4.32. The values of apparent resistivity collected in log steps were plotted on a vertical line log on Strater software to depict the comparison between resistivity and subsurface profiles (Figure 4.29 and 4.30) while the vertical electrical sounding curves were rotated 90° right to compare the subsurface profile and resistivity variation. In Figure 4.29, resistivity increases with depth up to some point within the Nairobi Trachyte and then it shows a slight decrease with depth. The borehole log description indicates that there are two distinct flows of Nairobi Trachyte; the lower flow is weathered. It is also notable that the resistivity is lower within the old land surface (OLS) and begins to increase with depth at the bottom of OLS; a case that the borehole log description confirms is a transition to the competent Ngong' basalts and Kandizi Phonolite.

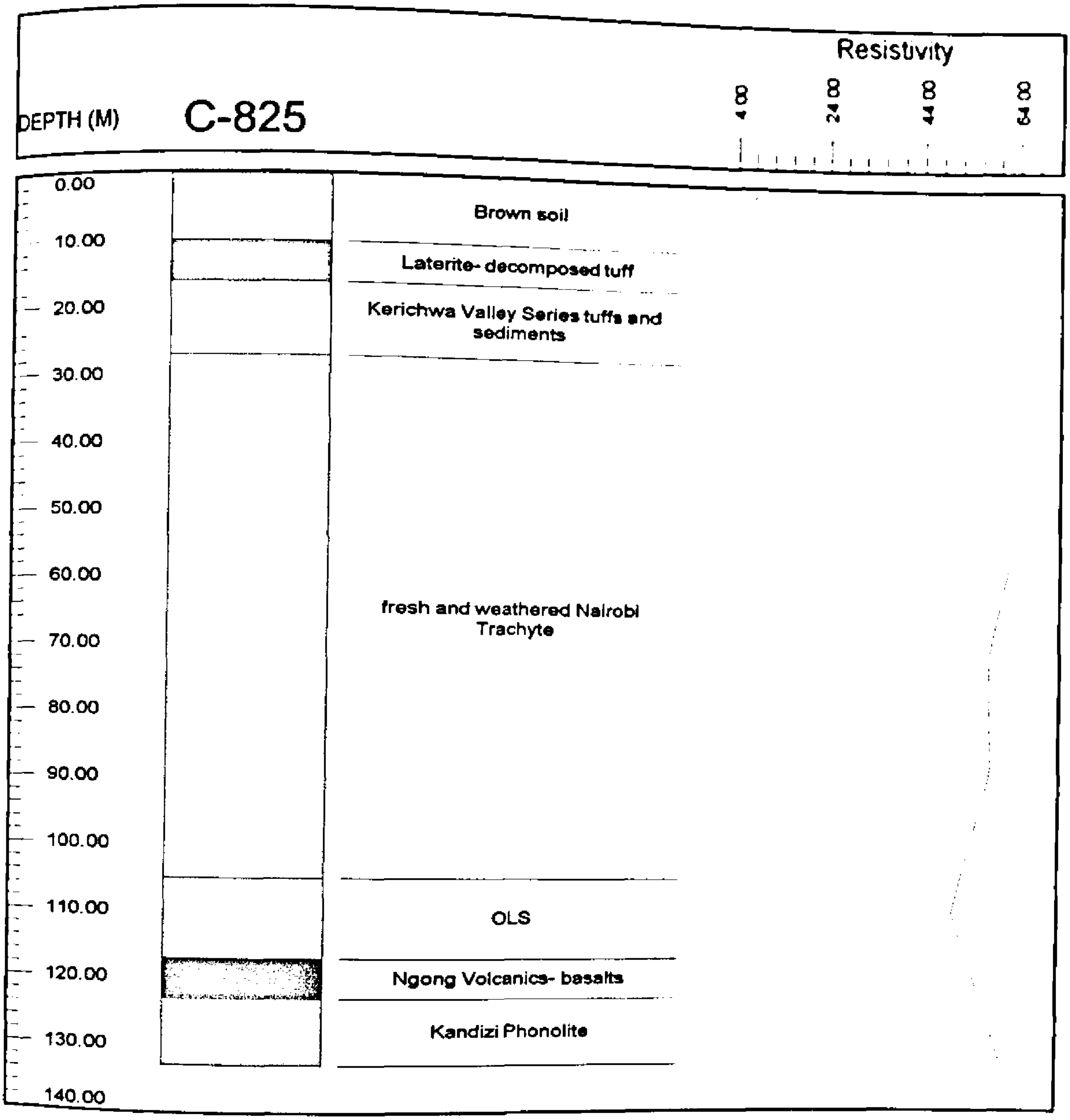

Figure 4.29: Subsurface profile and variation in resistivity with depth for Lenana School

In Figure 4.30, the resistivity increases with depth and remains almost constant within the Nairobi Trachyte and Mbagathi Phonolitic Trachyte showing that the strength properties of the two layers are close. However, it can be noted that the lower resistivity OLS layer between the two trachytes was suppressed.

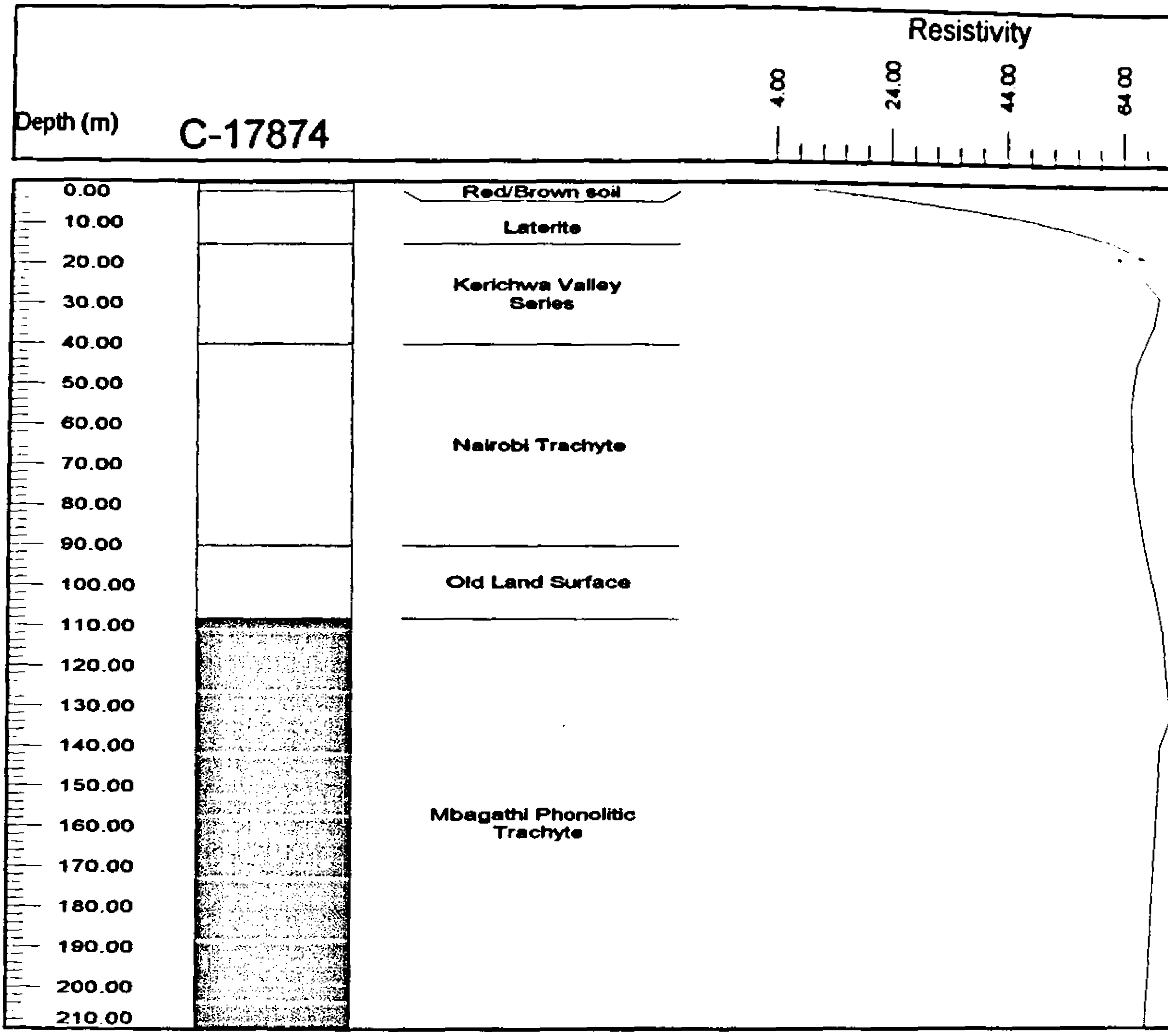

Figure 4.30: Subsurface profile and variation in resistivity with depth for Telkom Field at Kirandini

With a root-mean square value of 3.5%, the sounding curve in Figure 4.31 shows that the resistivity increases with depth. The layered resistivity model shows that it increases up to 77 m within the Mbagathi Phonolitic Trachyte and is high (111.5 Ω-m) within the upper part of the Athi Series. Then at 120 m depth, the resistivity drops from 111.5 Ω-m to 46.8 Ω-m. It can be observed that water is encountered at depths of 130 m and 170 m within the Athi Series as indicated in the two borehole profiles. The resistivity therefore drops as the groundwater is approached.

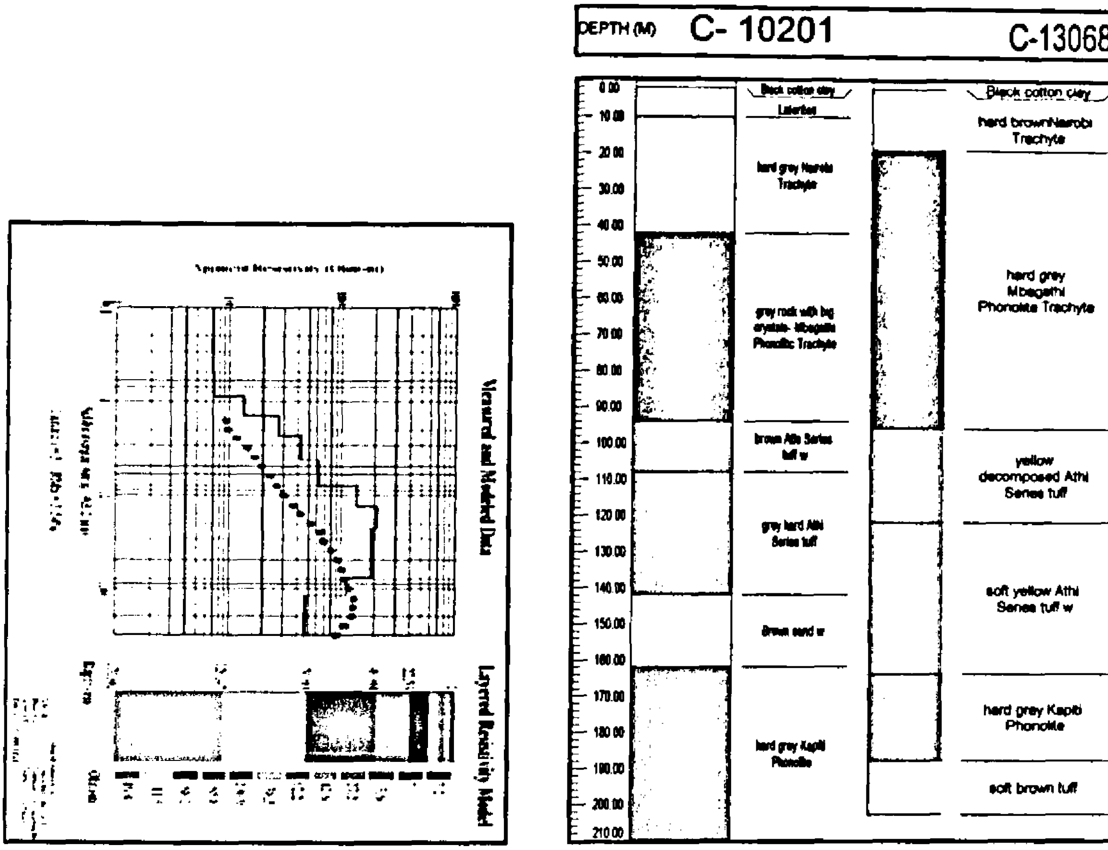

Figure 4.31: Comparison of a rotated sounding curve and subsurface profiles for Catholic University of Eastern Africa

The sounding curve in Figure 4.32 shows that resistivity reduces with depth and reaches a low of 4.1 ohm-m at 15 m depth before it begins to rise. This rise is within the lateritic gravel in borehole C-10497 in the vicinity of the site. The resistivity is high within the fresh Nairobi Phonolite which from geotechnical tests is known to be very strong. The resistivity drops within the weathered Nairobi Phonolite and Athi Series just at the bottom of the OLS at 84 m. From borehole records, aquifers were struck within the weathered phonolite and the Athi series formations below 84 m depth.

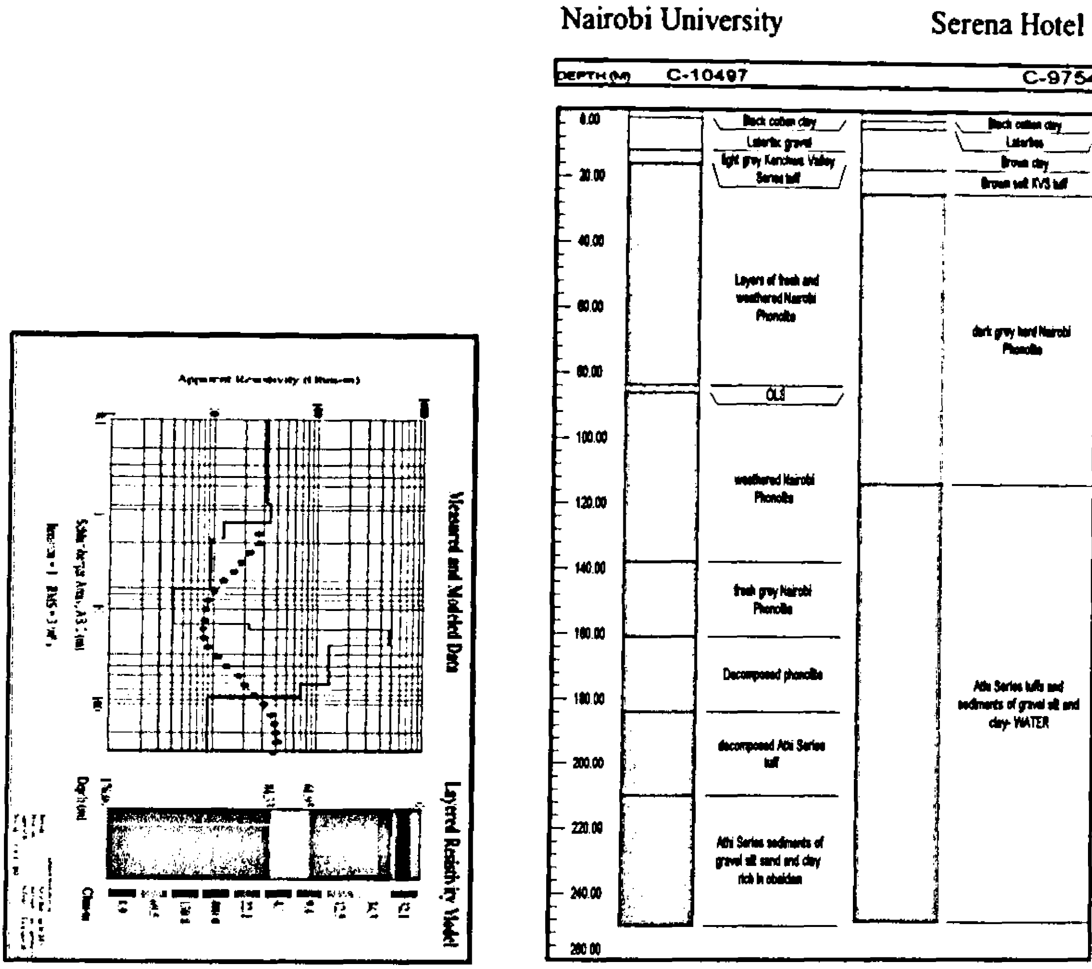

Figure 4.32: Comparison of a rotated sounding curve for Nairobi University and subsurface profiles for Nairobi University and Serena Hotel

Figure 4.33 presents correlation of a sounding curve and subsurface profile for nearby water supply borehole at Mirema Drive. The near-surface low resistivity layer that terminates at 13 m is underlain by a high resistivity layer that is shown as grey vesicular Kerichwa Valley Series tuff on the subsurface profiles. The bottom of the low resistivity layer corresponds with the levels of nearby rivers indicating saturation. Groundwater is found to flow at the interface between the soft overburden and weathered rock in other parts of the city. Geotechnical boreholes at Kasarani Complex and foundation failure investigation pits at Garden Estate show that the weak layers extend to between 6 and 8 m depth.

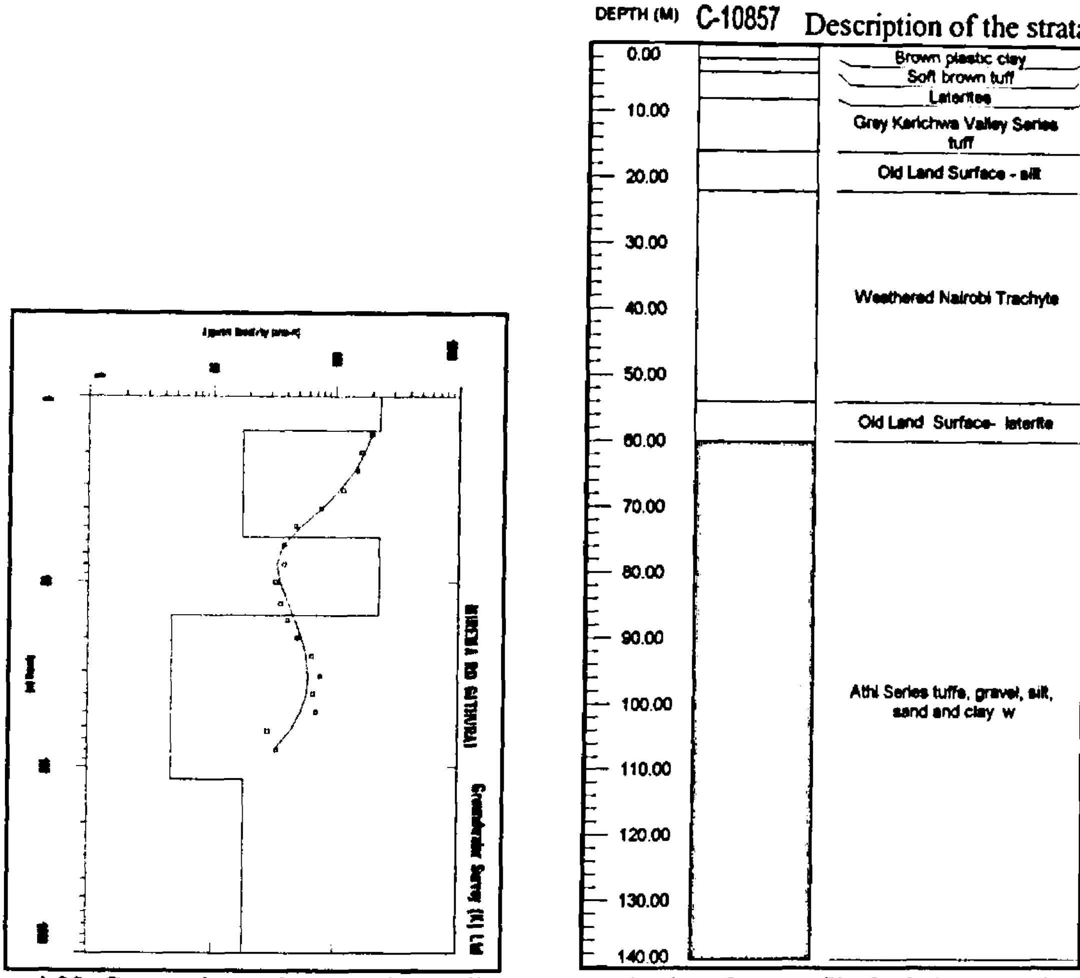

Figure 4.33: Comparison of a rotated sounding curve and subsurface profile for Mirema Drive, Githurai 44 (VS-48)

Figure 4.34 presents a sounding curve and subsurface profile at Pumwani Secondary School sports field. Resistivity of the three near-surface geoelectric layers increases with depth. In some logs from nearby boreholes, these layers are described as decomposed tuffs while in others they are described as laterites. The subsurface profile indicates that the laterites extend to 8 m depth and are underlain by weathered Kerichwa Valley Series tuff. The lower resistivity fourth layer between 10 m and 50 m in Figure 4.34 is a combination of Kerichwa Valley Series tuff and weathered Nairobi Phonolite. From records, the first (minor) aquifer in the borehole was struck at the contact between the Kerichwa Valley Series and Nairobi phonolite. Fresh Nairobi Phonolite is the high resistivity layer. It can therefore be concluded that it is possible to determine the depth to competent layer through resistivity survey.

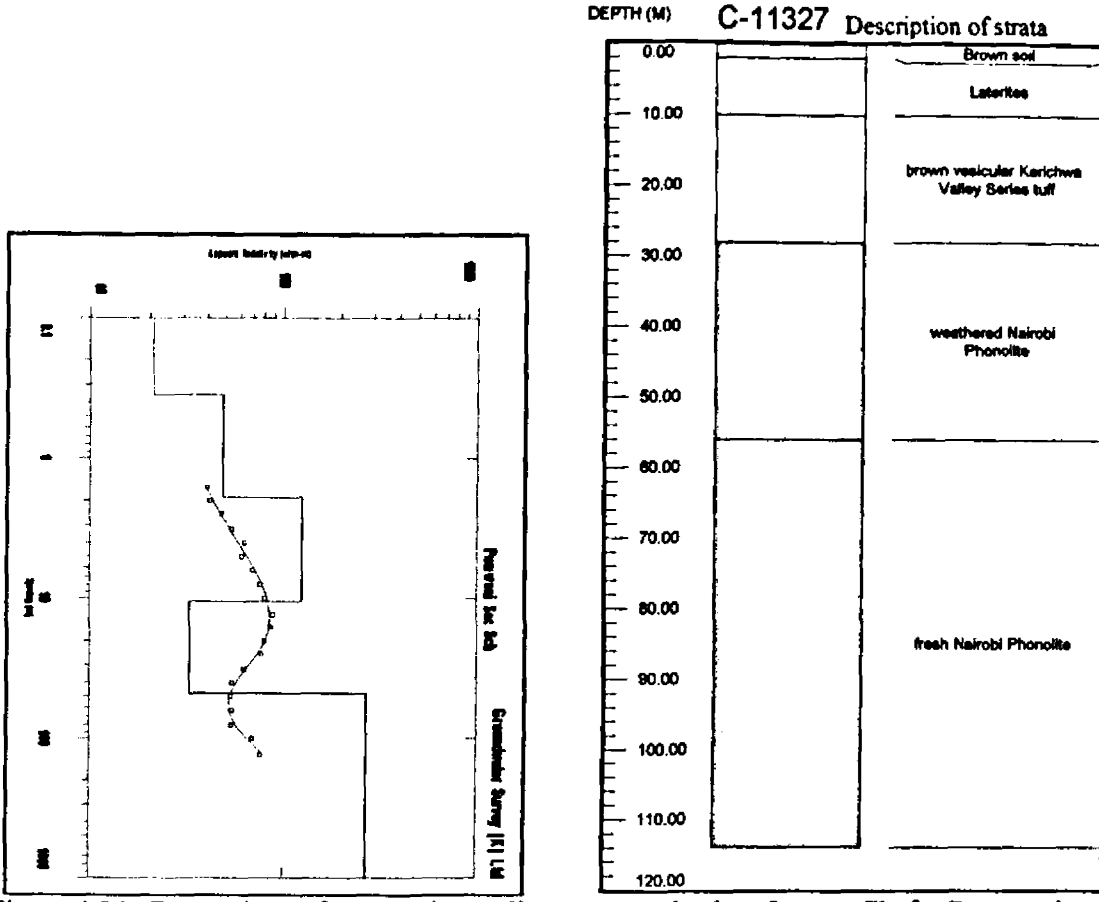

Figure 4.34: Comparison of a rotated sounding curve and subsurface profile for Pumwani Secondary School (VS-37)

The sounding curve and subsurface model at East Gate to National Park off Mombasa Road (Figure 4.35) show that a low resistivity layer commences at 52 m coinciding with the bottom of fresh Nairobi Phonolite encountered in the borehole at Express Kenya. The shape of this curve is similar to the ones for Madaraka and Old Airport Road where the bottom of the phonolite varies from 50 m to 100 m in boreholes. The thin weathered phonolite layer (boulder bed) sandwiched between two fresh phonolite flows is reported as one of the aquifers in most boreholes but is not detected on the resistivity curve.

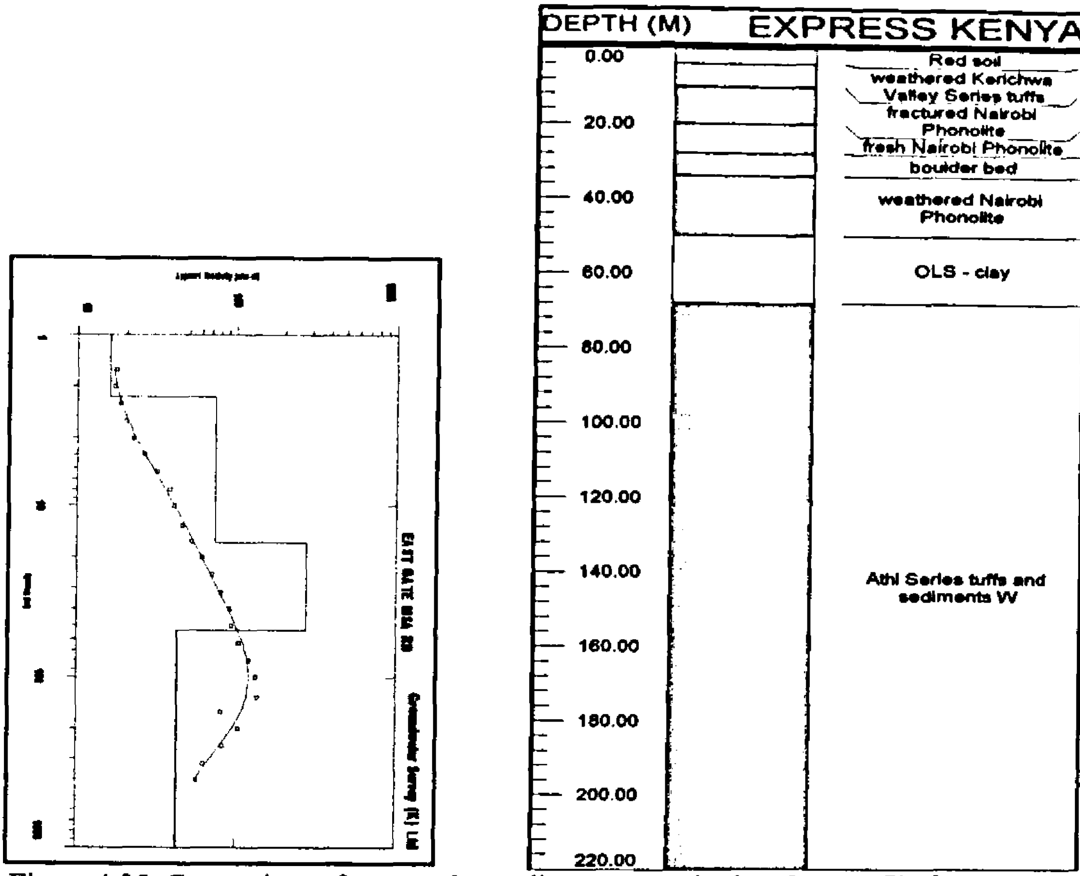

Figure 4.35: Comparison of a rotated sounding curve and subsurface profile for the East Gate to National Park (VS-39)

4.2 Geotechnical conditions

Subsurface profiles and project completion reports obtained from the Ministry of Roads and Public Works, consulting engineers and contractors indicate that the engineering character of the Nairobi subsoil varies quite significantly from place to place and even within construction sites. It is not possible to provide continuous stratigraphic profiles relevant in geotechnical terms for the entire study area.

This part of the study thus provides the qualitative and quantitative properties of subsurface materials encountered at some construction or failure investigation sites; position of the water table (where available); foundation type adopted and design pressures used. The sites described in subsurface characterisation are indicated with F- while shoring and underpinning sites are indicated with S- on the geological map in Figure 4.36. The legend of the geological map is shown in Figure 1.3. Because the City Centre sites are very closely spaced, Figure 4.37 was prepared for ease of reference. The results of the study are also presented in summary tables as well as in appendices.

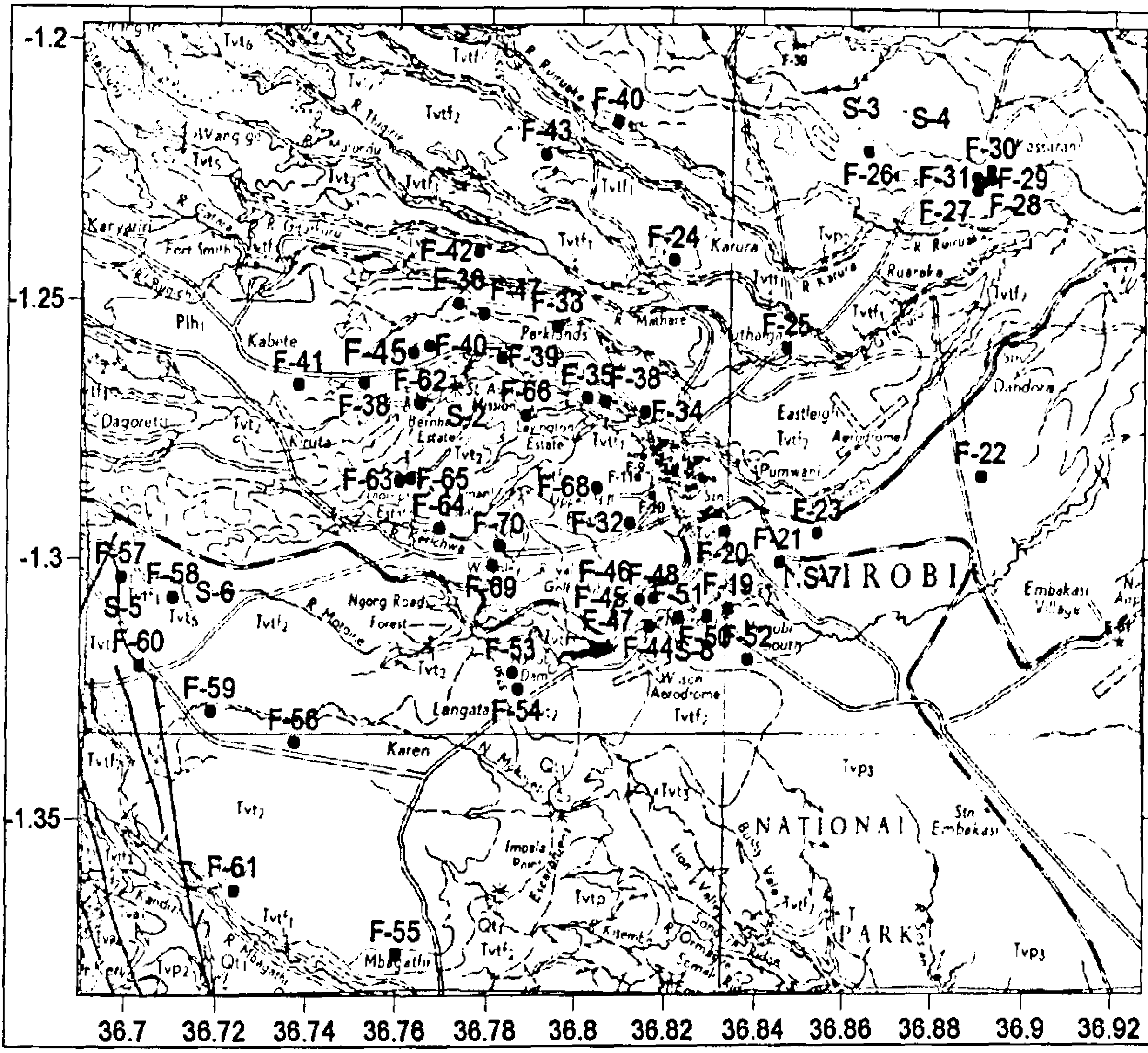

Figure 4.36: Subsurface characterisation and shoring and underpinning sites

(F-subsurface characterisation sites, S- shoring and underpinning sites)

4.2.1 Engineering properties and bearing capacities of critical subsoil layers at building sites and foundation types used

4.2.1.1 City Centre

The City Centre sites are indicated on Figure 4.37. Borehole profiles indicate that the subsurface materials beneath Nairobi City Centre vary from place to place and even within construction sites as will be shown here.

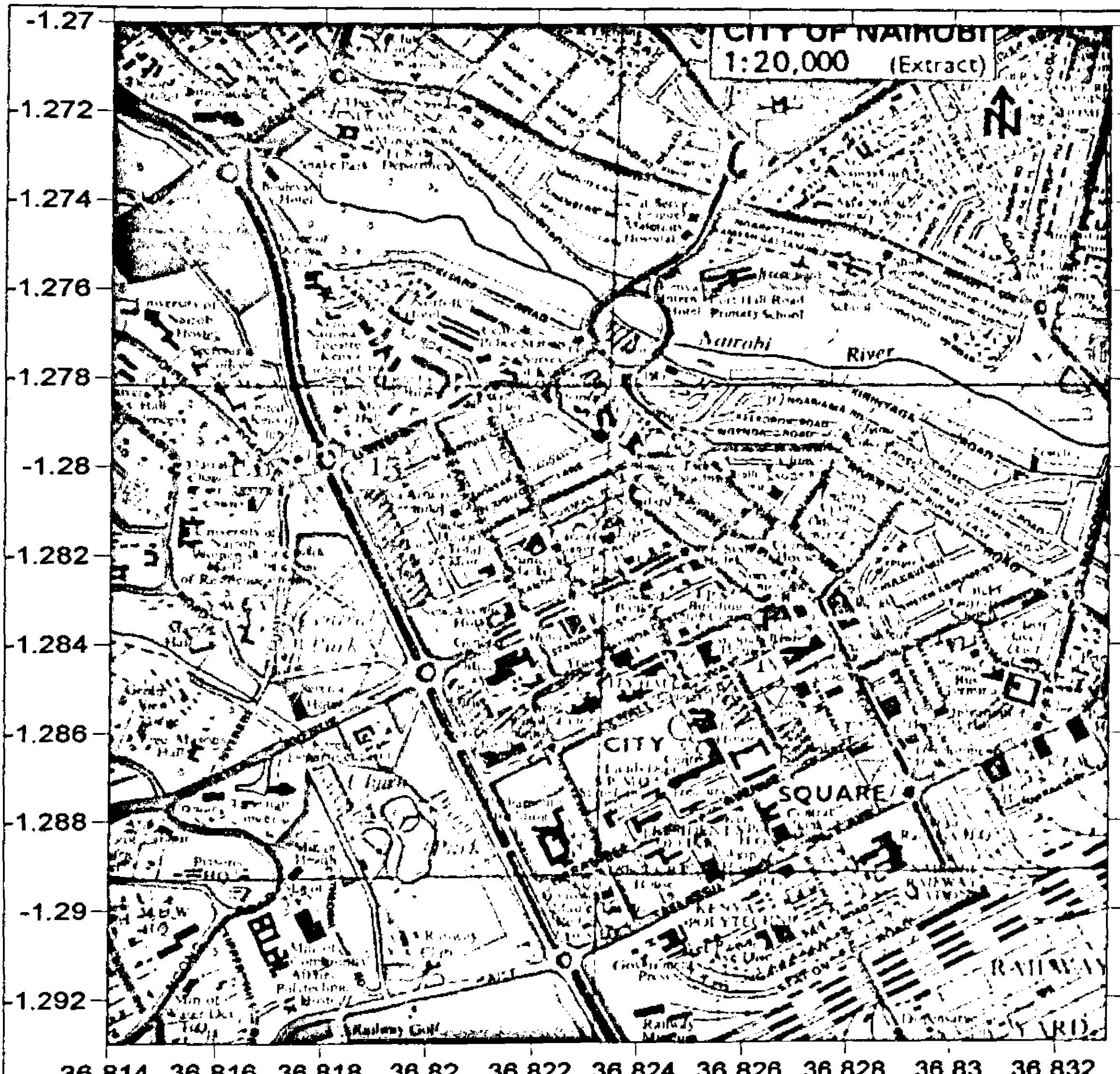

Figure 4.37: Subsurface characterisation sites at Nairobi City Centre (Survey of Kenya map)

i) Continental House, Six-eighty Hotel and Lutheran Church sites

Before construction of Continental House (F1), three geotechnical investigation boreholes were sunk up to maximum depth of 13.25 m. Water was struck in all boreholes at depths between 2.0 and 2.7 m. The ground profile consists of the strata shown in Figure 4.38. Crushing tests on selected samples of the core show that the rock varies considerably both in material quality and in the state of the rock mass as a whole. Continental House is supported on 8 m by 6 m grid of pad foundations with a suspended ground floor slab. The founding level is 4 m deep, on yellowish-grey tuff with a design bearing pressure of 400 kN/m^2 (Appendix 8 S/No. 1).

At Sixeighty Hotel (F2), six boreholes were sunk to a maximum depth of 15.4 m. Water was struck in all boreholes at depths varying from 0.9 to 1.1 m. Observations of the water level over a two week period resulted in fluctuations between 0.8 m and 1.3 m depth. The foundation conditions on the plot are as described in Figure 4.38. The agglomeratic tuff is mainly made of large angular phonolite boulders in a matrix of yellow tuff. The boulders are very hard but the matrix is so weak that the core samples are often very broken. On testing, most samples exhibited brittle failure with considerable variation in the elasticity modulus. Sixeighty Hotel is built as ten storeys above street level with one basement and is supported on pad foundations taken to rest on black tuff at 4.5 m with design bearing pressure of 800 kN/m^2 (Appendix 8 S/No.2).

Lutheran Church (F16) subsurface consists of the strata shown in Figure 4.38. At 6 m to 9 m depth, the clay content range is 62-79% and soil tests yielded liquid limit results ranging from 91-105% and plasticity indices of 61-71. All these plotted as extremely high plasticity soils on the plasticity chart (Appendix 6 (c)). According to the Casagrande classification of fine-grained soils, these soils can be termed as very unusual soils. Oedometer tests indicate that the soils settle when loaded under natural moisture content and when flooded under load as shown in Table 4.5. The "old" St Andrews Church (F17) built in 1910 is founded on this deep alluvium. An underground river within this site flows into Nairobi River. By 1930, there was concern as to whether the St Andrews Church building was in a state of collapse, but it is still standing thanks to buttressing and repair work. In 1964, the church was found to have settled by approximately 0.2 m to 0.5 m (Ministry of Works, 1969)

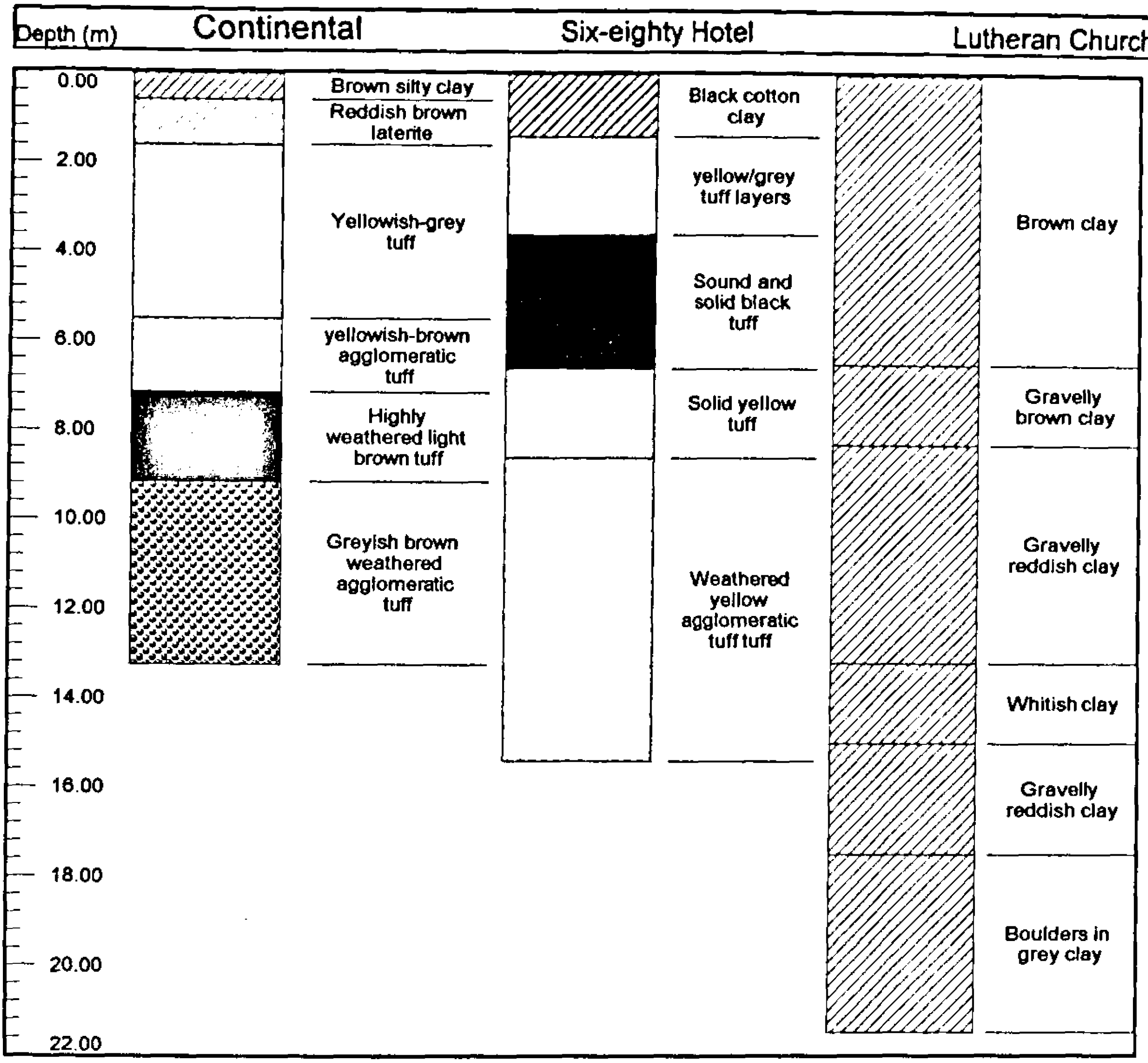

Figure 4.38: Subsurface profiles at Continental House, Six-eighty and Lutheran Church

ii) Lillian Towers and Tom Mboya Street sites

Excavation for construction of Lillian Towers (F3) encountered very variable conditions with soft and wet spots of yellow tuff decomposed to clay. Five geotechnical boreholes were drilled commencing at the bottom of the excavation at 3 m. The profiles vary from borehole to borehole but the summarised log is as shown in Figure 4.39. Settlement test results on samples at 2.9-3.2 m and 13-13.3 m indicate that they could undergo settlements of 18-21 mm and 36-44 mm when flooded and loaded with 200 kN/m² and 500 kN/m² respectively, as shown in Table 4.5.

The water supply well at Lillian Towers has the following strata starting from the ground surface: 0-4 m brown soft laterite, 4-6 m black soft tuff, 6-10 m brown laterite, 10-12 m black fine silt with clay, 12-16 m black soft tuff with clay, 16-26 m dark grey phonolite. The fourteen-storey Lillian Towers structure with two split level basements is supported on 7 m deep pad foundations resting on yellow tuff with allowable bearing pressure of 500 kN/m² (Appendix 8 S/No 3).

Table 4.5: Summary of settlement values for some building sites

Site	Depth (m)	Settlement (mm)					
		100 kN/m²		200 kN/m²		500 kN/m²	
		NF	F	NF	F	NF	F
Lilian Towers	2.9 13.3	14 9	16 16	17 18	18 21	26 39	36 44
Lutheran church	5.6	27	32	33	42	42	47
KIRDI	1.5	24	26	28	38	40	52
Onyonka	1.2	25 20	36 25	36 26	61 42	50 45	84 72
Ngei	1.8	17	DNA	22	DNA	25	DNA
Adams Arcade	3.0	28	DNA	39	DNA	49	DNA
Sonning Road	1.3	8	28	10	29	12	40
Kileleshwa		18 16 40 21	22 21 78 24	23 21 62 31	42 42 92 42	30 45 105 42	70 61 150 70
Kenya Soil Survey	1.2	25	62	56	88	120	128
Kangemi United Club	1.5	40	DNA	56	DNA	108	DNA
Langata barracks	1.5		42		58		84
Government Sec Col.	1.3	10	18 48	25	36 80	40	64 116
Dandora Water Supply	1.0	10	17	15	22	24	27
Athi River	1.2	DNA	58	DNA	74	DNA	79

(F- Flooded, NF- non-flooded, DNA- data not available)

Construction of an eight storey structure on Plot LR 209/503 (F4) along Tom Mboya Street encountered very variable soil conditions during foundation excavation such that it was very difficult to implement the structural designs that were submitted to the City Council. The contractor requested for a site investigation commencing at the bottom of the excavation at 3.6 m. Three boreholes were sunk to a maximum depth of 19.5 m. The borehole logs show a succession similar to Lillian Towers site but with a completely decomposed yellow tuff at 3.7 –

5.1 m depth. The pad foundations of the building are set on a 2 m thick black tuff that acts as a raft spreading maximum pressures of 100 kN/m² on the decomposed yellow tuff.

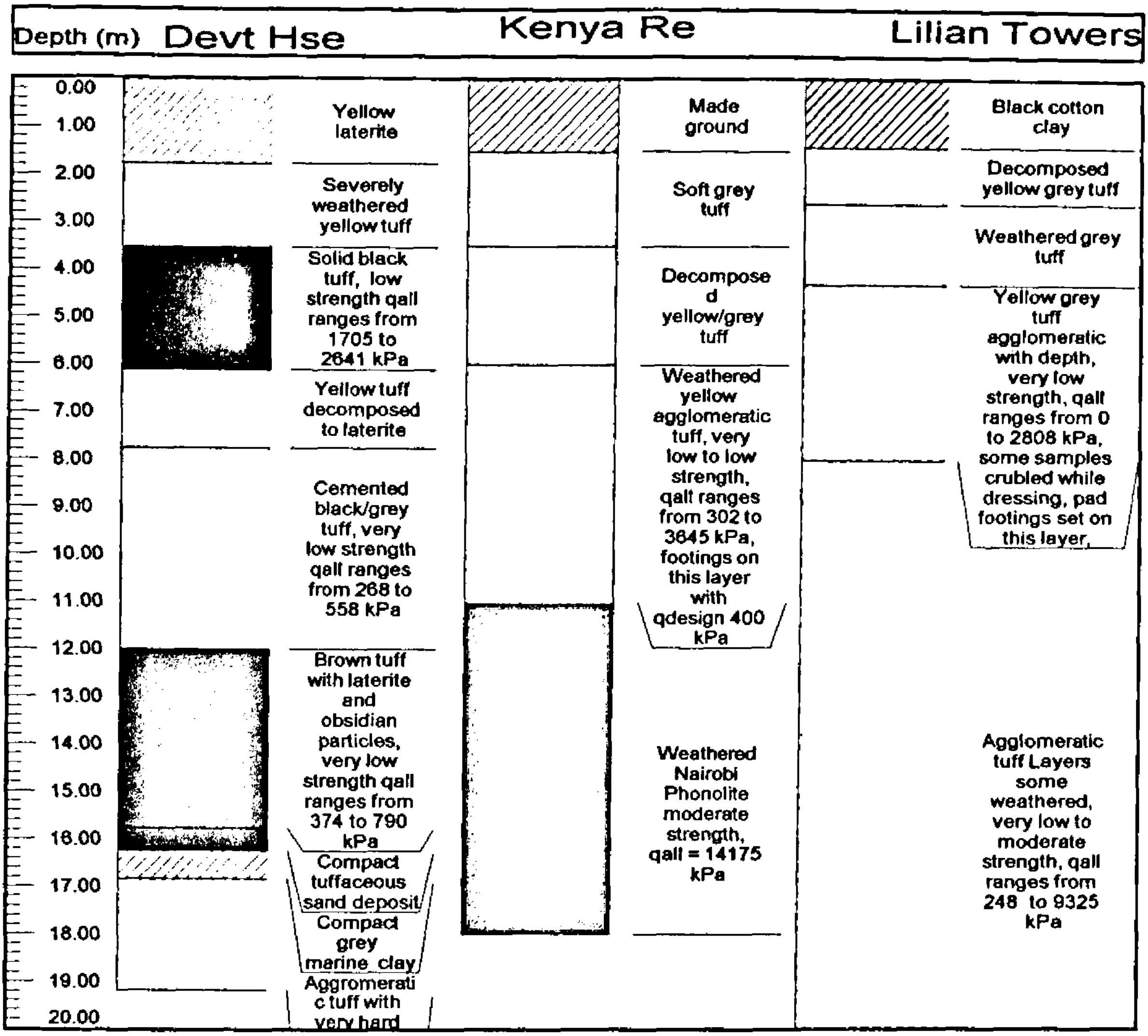

Figure 4.39: Profiles for Development House, Kenya Re and Lillian Towers sites

iii) Development House, Union Towers and Sonalux House sites along Moi Avenue

At Development House Phase II (F6) site, five boreholes were drilled to a maximum depth of 18 m and a summary of the profile is presented in Figure 4.39. No water was struck in any borehole though the vicinity of the site is known to have shallow groundwater. For instance, at the Coffee Board of Kenya building, water was encountered at 1.1 m depth. The allowable bearing capacity of the decomposed material at 3.6-6 m was found to be in range of 120-180 kN/m² and was thus ruled out for supporting foundations. Founding levels were recommended on yellow agglomeratic tuff at 7-8 m with design bearing capacity of 500 kN/m². This was despite the fact that at this depth, some pockets of weak tuff were found at random. Development House phase II is supported on pad footings on yellow agglomeratic tuff at 6 m with design bearing pressure of 400 kN/m² (Appendix 8 S/No. 6).

It was also recommended that the Development House Phase I and Church House are shored with strong retaining walls. This could be done by excavating for basement in small amounts and placing the retaining walls step by step as the excavation continued. The walls were to be placed in 2-3 m long units with shorter units at the towering lift portion. This type of construction was necessary to ensure that the walls of foundations of the existing structures were disturbed in bits not as a whole. For the lower parts of Church House, a 0.5-0.8 m thick retaining wall (tapering upwards and ribate-keyed at the bottom into the rock) was recommended. The site investigation geologist interpreted that F6 site lies at the transition zone where the Nairobi Phonolites meet with the middle and the lower Kerichwa Valley Series tuffs.

The subsurface profiles at Union Towers and Sonalux House differ from the Lillian Towers profile and consist of the following: 0-1 m decomposed tuff and clay, 1-2.5 m weathered black tuff decomposed to cobles at some points, 2.5-6.7 m layers of fresh/ broken/ weathered and decomposed tuff, and, 6.7-20 m layers of severely weathered and decomposed agglomeratic tuff. Given the weathered nature of the subsurface, the pad foundations for Union Towers were taken to 3 m depth to rest on black tuff with a design bearing pressure of 250 kN/m² whereas the foundations supporting Sonalux House are designed as strip footings resting on brown weathered tuff at 6.5 m depth with a design bearing pressure of 150 kN/m (Appendix 8 S/No. 4).

iv) Kenya Reinsurance Plaza site and other nearby sites

The subsurface materials encountered at the Kenya Reinsurance Plaza site (F7) were described as "differing quite significantly from the general trend of materials encountered in other parts of the city". The succession and bearing capacities of the Kenya Reinsurance Plaza subsurface are given in Figure 4.39. The black tuff that supports most of the tall structures in the City Centre was not encountered at this site. Furthermore, the materials extending from the surface up to 8.3 m depth were found to be very porous and capable of transmitting water to the foundation bases.

It was recommended that the drainage system be designed to collect ground water into a central pool at lower levels and a pump to be acquired to pump water out into the city storm water drainage system. Such pumping would be vigorously active only during rainy seasons. The pad foundations for Reinsurance plaza building are set on yellow agglomeratic tuff at 6-7 m depth (Appendix 8 S/No. 7).

The nearby 28-storey Kenyatta International Conference Centre (KICC) building with two basements is supported on a raft foundation resting on the agglomeratic tuff. The subsurface conditions at the KICC (F12) are described as uniform and weathered up to 15 m (Appendix 8 S/No. 9). Investigation at Hilton Hotel site encountered yellow plastic clay of variable thickness underlying the black tuff sheet at 7 m depth. To avoid the expected differential settlement, Hilton Hotel is supported on a circular raft resting on agglomeratic tuff (Appendix 8 S/No. 10). The subsurface profile at the International Life House indicates that the rock is weathered up to 18 m and soft or fractured up to 30 m.

Real-time settlement measurement at 31 different points for a period of 417 days (April 1970 to May 1972) with a Bench Mark at the Jogoo House A shows that the KICC building underwent heave and settlement. Maximum heave of 4 mm and maximum settlement of 13 mm were recorded. Some points did not record any settlement though in the intervening period they showed slight heave. Real-time settlement results from Bruce House along Standard Street show settlement values ranging from 13 mm to 18 mm. These results together with those taken at KICC indicate that the weight of the skyscrapers, if well constructed may not cause intolerable settlement of the Nairobi subsoil unless the conditions alter.

The ground strata at National Housing Corporation (NHC) building along Harambee Avenue (F13) include the following: 0-2 m black cotton soil; 2-4 m decomposed yellow tuff; and, 4-6 m dark grey vesicular tuffs. The decomposed yellow tuff was found to be expansive when in contact with water. It was recommended that the foundations by-pass the yellow tuff and be supported on the grey tuff with a drainage system instead of a watertight basement structure. The neighbouring 6-storey Javan Bharat building was recommended for underpinning. NHC building is supported on pad footings resting on grey tuff at 4.5 m depth with design bearing pressure of 500 kN/m^2 (Appendix 8 S/No.11). The nearby 27-storey Cooperative House is also supported on pad footings resting on the grey tuff at 4.5 m with a bearing pressure of 400 kN/m² (Appendix 8 S/No. 12).

v) Nation Centre and Kimathi Street sites

At Nation Centre (F5), four boreholes were drilled to a maximum depth of 24.05 m on the lanes outside the footprint of the site. The consultant had requested the investigation to indicate possibility of setting foundations at 11 m. Two borehole logs show a very weak zone at 8 m to 11 m while the other two have the weak zone at 9 m to 12 m. In the report, it was noted that the requested founding level was within the weak zone. Because of local pockets that were expected to cause differential settlements of 60 mm for pad foundations, the strength of the weak zone was difficult to predict. Besides, the brown agglomeratic tuff below 12 m exhibited clay-filled diagonal fractures and in some places was completely decomposed to gravelly clay. It was recommended that the foundations be set at 12 m with allowable bearing pressure of 400 kN/m² and the excavation of the foundation be inspected in time to allow for any adjustment of depth and/or width of footings.

Express Transport Building site along Kimathi Street has a succession similar to that at Lillian Towers but the brown tuff and marine clay are missing. The six-storey Express Transport building is supported on pad foundations resting on black tuff at 4.2 m with an allowable bearing pressure of 400 kN/m² (Appendix 8 S/No. 5). A similar succession was encountered at Lonrho House and it was recommended that loads be supported on yellow tuff at 5-7 m depth with allowable pressure of 500 kN/m² provided that the diagonal cracks are sealed during foundation placement.

vi) Loita House and nearby sites

Loita House (F8) site investigation sunk five boreholes to a depth of 33 m and the sixth borehole to a depth of 75 m. Water was encountered within the ashy tuff decomposed to black sand at depths between 1.5 and 5.5 m in all boreholes. Permeability tests carried out in all six boreholes indicate a coefficient of permeability (K) of the order of 10^{-6} -10^{-7} cm/s. The amount of sulphates and chloride in the groundwater were found to be high at this site. In addition, the agglomeratic tuff layer at the site was found to soften and generally become unstable when in contact with water.

It was recommended that sulphate resistant cement be used for the part of the structure below the water table. Chloride could also lead to corrosion of steel structures and reinforcement, so corrosion resistant coatings were recommended. Due to the fractured condition of rock strata up to 35 m observed on the core logs, it was recommended that the sides of all deep excavations on highly weathered agglomeratic tuffs and soft rocks be adequately supported. On harder, less weathered phonolite, no form of support was recommended unless the rock is heavily fractured and/or faulted in which case rock anchoring and bolting would be used.

The profile at the nearby Laico Regency Hotel consists of 6.5 m thick brownish clay, 2 m thick weathered tuff, 11.3 m thick agglomeratic tuff followed by phonolite. The brownish clay is 3 m deep at Chester House and 2.1 m deep at Herani House. The weathered tuff underlying the brown clay is 7 m thick at Herani House. Geotechnical information from the adjacent Nyati House is similar to that at Loita House.

The subsurface profile at the 12 storey (with two basements) Utalii House indicates existence of 5 m thick plastic clay followed by weathered material up to 15 m. Utalii House is supported on pad foundations taken to yellow agglomeratic tuff at 6-7 m depth with a design bearing pressure of 300 kN/m² (Appendix 8 S/No. 13). The 29 storey Nyayo House a short distance away encountered very soft materials up to 11 m and is supported on a cellular raft resting on black tuff at 3 m with a design bearing pressure of 350 kN/m² (Appendix 8 S/No.14).

vii) Sites west of Uhuru Highway

Investigation for Central Park Monument (F9) revealed the following properties for a clay layer extending from the ground surface to 6 m: LL= 61-82%; PI = 31-39%; linear shrinkage = 15-20%; clay content = 43-71%. The soils plotted as clays and silts of medium to high plasticity as shown on the charts in Appendix 6. The consolidation tests revealed swelling soil at 1.5-2.5 m and 5.5-6 m while the samples at 2.5-3 m and 4.0-4.5 m though clay underwent neither swelling nor collapse. The values of the potential hydrocollapse strain ranged between 0.01 and 0.035 (Table 4.6) indicating moderate trouble according to the criterion provided by Coduto (2001) presented on Table 3.3. To avoid the potentially large total and differential settlements of the soft clay, raft foundations were recommended.

Table 4.6: Potential hydrocollapse strain ranges for some sites

Site	Soil type	Potential hydrocollapse strain range	Classification (Coduto, 2001)
Central Park	Brown clay	0.01-0.035	Moderate trouble
Madaraka Pri	Black silty clay	0.142- 0.198	Severe trouble
Kongoni Pri	Silty clay	0.19-0.210	Severe to very severe trouble
Kabete Armed Forces	Red clay	0.08- 0.815	No problem to very severe trouble

Investigation for the Presidential Dais at Uhuru Park (F10) established that the soft clay extends to 8-8.5 m depth. Borehole 3 located immediately west of the central structure encountered very soft formations that were found to be subject to very large settlements if loaded. Pile foundations were recommended for this site. The subsurface formations at TPS Serena Hotel (F11) are as shown in Figure 4.40. Black tuff was not encountered in BH 6 between the plot and Uhuru Highway and the plot was thus considered to be situated at the edge of the tuff formation that underlies most of the City Centre. The black tuff was also not encountered in any of the boreholes at Uhuru Park.

Serena Hotel was proposed as a 22 storey building but only five were constructed. The construction report indicates that an easterly slope was encountered with enormous amounts of groundwater flowing that no basement could be accommodated without experiencing lateral pressures. The structure is supported on strapped pad foundations resting on tuff at 6 m depth with a design bearing pressure of 160 kN/m² (Appendix 8 S/No. 8). The low design pressures used were to take care of possible weakening of foundation soil by the flowing groundwater.

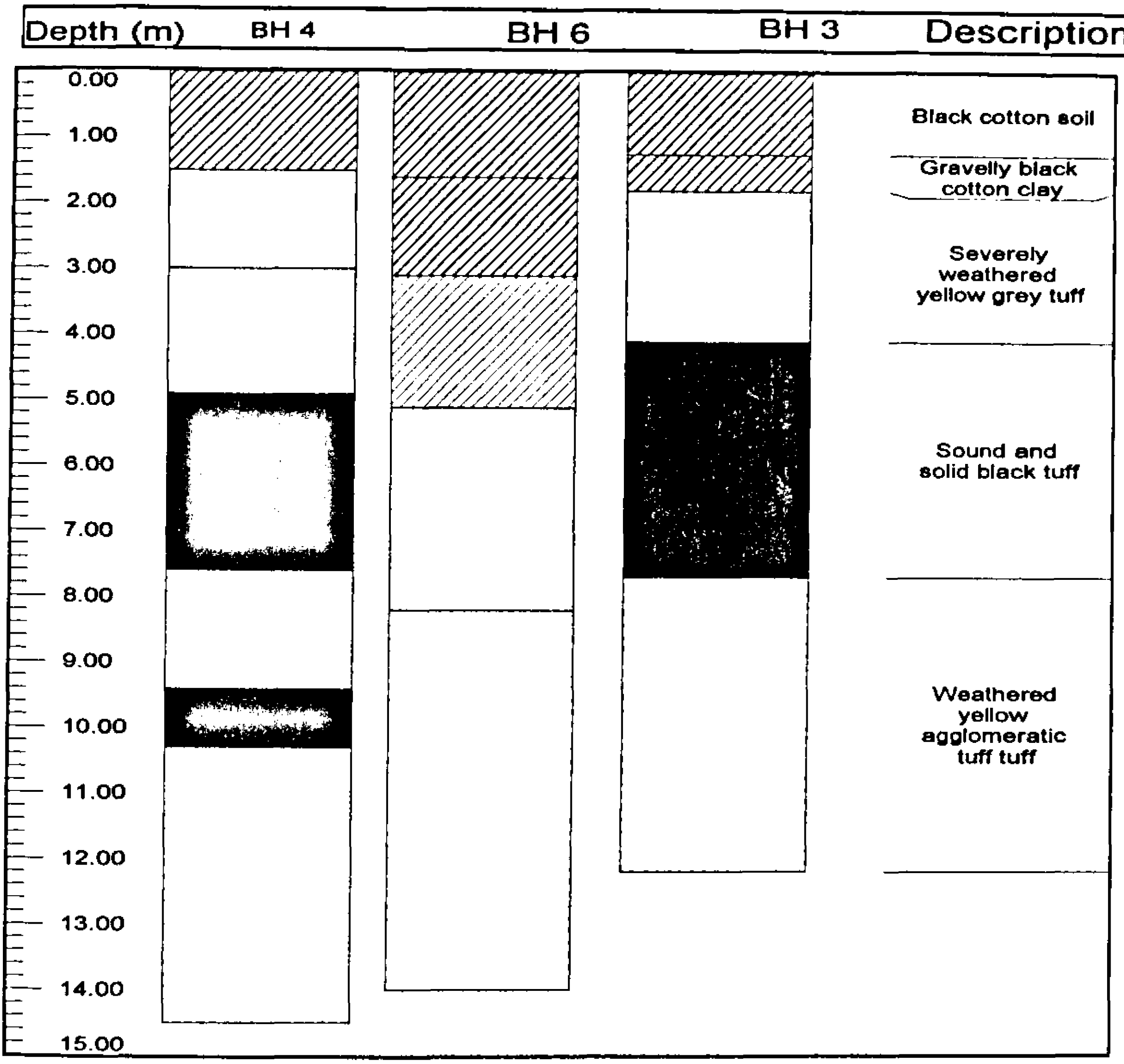

Figure 4.40: Subsurface profiles at TPS Serena Hotel

viii) Nginyo Towers and Anniversary Towers sites

Construction of the 10-storey Nginyo Towers along Koinange Street (F14) considered information that many tall buildings within the vicinity had encountered underground rivers flowing towards Nairobi River. Geotechnical investigation carried out at the site indicates that the subsoil profile is variable and includes a layer of highly fractured pervious rock with a lot of groundwater flow. The bearing capacity tests indicate very low allowable pressures at some spots. Because of the variable profile, it was determined that if customary pad foundations were adopted, the structure would be subject to differential settlement. Also, a shoring system was needed to support the adjacent foundations that would be exposed during excavation for basement.

The foundation concept adopted in view of highly fractured and pervious materials of variable profile was to provide an interconnecting grillage of pad footings with short spans. This particular approach led to a foundation solution that proved simple and repetitive to construct, whilst acknowledging the relatively uncertain conditions and the need for an economical construction. To control the groundwater flow into the excavation, a system of containerising water was developed. This was to allow the basement slab to be cast on pervious rock and the water flowing in its natural course was allowed to pass below the heavily waterproofed slab. Since the existing foundations were almost at the same level with new foundation, they were shored with 45° props that were removed as the work progressed. No damage to the existing structure was reported.

Designers and builders of the Anniversary Towers along University Way were very conscious of the subsurface conditions encountered at sites within the vicinity. Furthermore, the structure was bordering shallow foundation structures that had to be supported during excavation and subsequent construction. To avoid ground movement, perimeter reinforced cantilever walls were constructed to support the structures. They also served the purpose of excluding groundwater that was associated with distress in a high-rise structure within the vicinity. Strip and pad foundations combined with the floor slab and resting on fractured rock were utilized at this site. Trenching was done to pump out groundwater that was much less than expected based on reports from the adjacent sites.

4.2.1.2 Industrial Area

i) KPC, Mitihani House and General Motors sites

Investigation for the foot bridge near General Motors encountered the succession shown in figure 4. 41. Profiles at Kenya Pipeline Company headquarters and Mitihani House in South C indicate that the subsurface is variable and consists of plastic clays, weathered tuffs and some intact layers. Site investigations at Morris & Company site along Mogadishu Road, Kenya Commercial Bank at Industrial Area, and National Water Conservation and Pipeline Corporation site along Dunga Road encountered profiles similar to that encountered at KPC site.

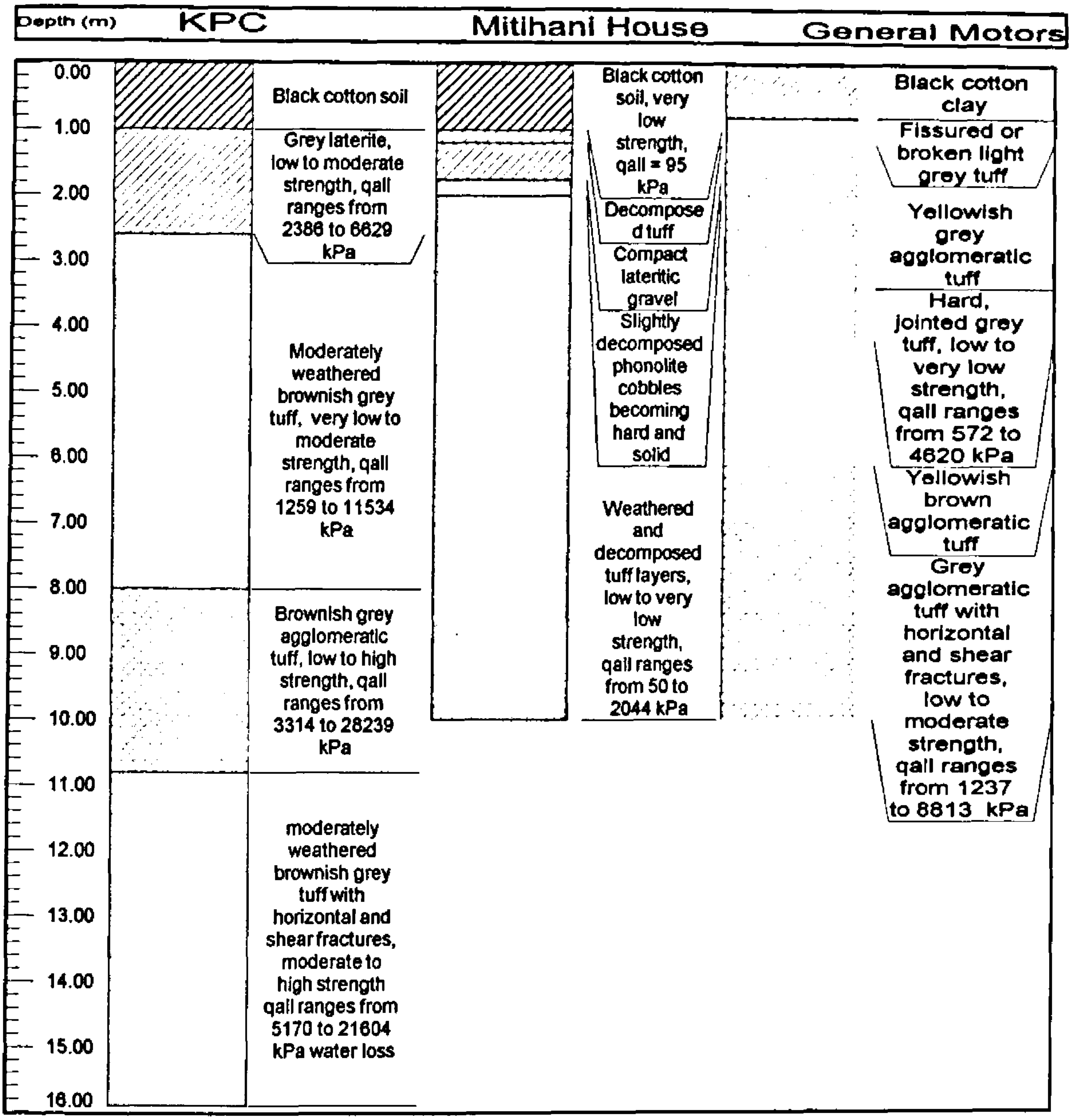

Figure 4.41: Subsurface profiles for KPC, Mitihani House and General Motors sites

Eight trial pits excavated at the Kenya Industrial Management Services site at South C provide a succession similar to that encountered at Mitihani House Site. Oedometer tests on the clay samples show that the soils will settle by 28-37 mm under 200 kN/m² pressures and by 40-50 mm under 500 kN/m² pressures (Table 4.5).

ii) **Enterprise Road site**

Construction of a six-storey commercial building along Enterprise Road commenced during the period of this study. Groundwater was encountered at 0.5 m depth at the site. The foundation was designed as combined slab and pad system set at 3.7 m depth with an average pressure of 400 kN/m^2. Considering the 3.2 m of submergence, the excavated overburden and the settlement that the ground had been subjected to in the past by the demolished structure, the effective pressure was considered to be slightly less than 300 kN/m^2. The basement structure was constructed after a general excavation was made with the help of diaphragm walls incorporated as retaining walls for the basement. French drains inside and outside the wall led water to a sump where it was pumped out. This design was influenced by the low level of applied loading and the need to prevent upward seepage of water through the slab.

iii) **Manchester Outfitters site**

In July 1970, there was collapse of a wall at Manchester Outfitters along Funzi Road (F21) injuring two people. The failure investigation revealed the following: the wall was supported on concrete strip footing at 0.25 m depth; the foundation was resting on fill; and, a 0.5 m deep trench had been excavated adjacent to the wall for construction of a building. Samples collected during the investigation were tested for Atterberg limits and grading. The test results indicate that the foundation soil has medium to high plasticity (Appendix 6 (b)) and grading curves show that the soil can be classified as silty sand (Table 4.7).

It was concluded that with the weight of the wall placed close to the excavated face and eccentric on the foundation strip, foundation stresses became too large and the soil underneath started yielding. This caused the wall to tilt which further increased the foundation stresses and foundation failure took place. The failure was promoted by the faulty foundation. The foundation should not have been placed on fill, or the subsoil should have been compacted first.

It is also important to note that strip footings must have sufficient width to provide an adequate bearing area to support the imposed loads on the ground support concerned. The depth of the footing is also important in terms of the ability of the foundations to resist local fluctuations in the bearing capacity of the ground without developing fractures.

Table 4.7: Classification of soils from the grading curves and plasticity charts

Site	Soil class	Plasticity term	Group symbol (BS 5930:1981)
Manchester outfitters	Clayey sandy SILT	High	CH
KIHBT	Silty CLAY	Intermediate to high	CI to CH
NSSF Karen	Clayey silty SAND	Intermediate to high	CI to CH MI to MH
Country Bus station	Silty CLAY	Intermediate to extremely high	MI to MV
Kileleshwa	Silty CLAY	Intermediate to high	CI to CH
Univ. of Nairobi Sports Field	Clayey sandy SILT	High	CH
Onyonka Estate	Silty CLAY	High to very high	CH to CV
Garden Estate	Silty CLAY	Very high to extremely high	CV to CE
Sonning Road Kibera	Clayey SILT	Intermediate to high	MI to MH
Site and Service Scheme	Clayey sandy SILT	Intermediate to high	MI to MH
Kyuna	Sandy SILT	High	MI to MH
Westlands Motors	Sandy clayey SILT	Intermediate to high	MI to MH
Lutheran Church	Silty CLAY	Extremely high	CE
GSU Ruaraka	Silty CLAY	intermediate to extremely high	CI to CE
PSC Club	Sandy clayey SILT	High to very high	CI to CV
Kitisuru NSSF Estate	Clayey SILT	High	MH
Kazora Flats	Silty CLAY	High	CH
Mathare 4A	sandy clayey SILT	Intermediate to high	CI to CH
Madaraka Pri. Sch.	Silty CLAY	Very high	CV
Kabete Armed Forces	Silty CLAY	High	MH

iv) Kenya Institute of Mass Communication and Kenya Institute of Highways and Building Technology

At the Kenya Institute of Mass Communication (F19), the grey tuff encountered at KPC site occurs at 0.8-1.4 m. The upper part of the rock is severely weathered. At some spots, black cotton clay overlying the grey weathered tuff is up to 2 m thick. Water encountered at 1.8 m depth quickly filled the holes after emptying. Allowable foundation pressures on the tuff were taken as 400 kN/m². Suspended foundation was recommended in the section with deep black cotton clay.

Kenya Institute of Highways and Building Technology (KIHBT) (F20) ground strata consist of 0.95-1.45 m thick black cotton clay, followed by agglomeratic tuff mostly consisting of large boulders up to 0.5 m wide in a matrix of poorly cemented yellow tuff. The top of this tuff layer is partly weathered to clay. In some holes where the agglomeratic tuff is not encountered, yellow tuff with a few clay seams and boulders are found. The plasticity chart indicates that the clays are uniform with medium plasticity (Appendix 6 (c)) and from grading curves the soil can be classified as silty clay (Table 4.7). Allowable pressure on the agglomeratic tuff was recommended as 250 kN/m².

v) National Social Security Fund (NSSF) South B Estate

Investigation for construction of NSSF South B Estate drilled nine boreholes to a maximum depth of 10 m. Being a large area, core samples indicate no correlation in the subsurface profiles encountered in boreholes. However, the summarised subsurface profile is similar to that encountered at General Motors site in Figure 4.41. One borehole (No. 9) encountered loose saturated materials up to 9.6 m. Houses constructed in the vicinity of borehole No. 9 site are supported on point bearing piles whereas the rest of the houses are supported on pad footings.

4.2.1.3 Eastlands Area

Construction of Tena Catholic Church (F22) encountered 0.9 m of black cotton clay followed by yellowish-brown moderately weathered tuff. Plate loading test indicates that a bearing pressure of 300 kN/m² on 1.2 m strip footing would cause a maximum settlement of 25 mm on the tuff. Geotechnical site investigation for a two storey structure at the Church Army Academy along Jogoo Road (F23) revealed that groundwater exists at 0.6 m depth, at the interface between black cotton soil and tuff. During construction, all the black cotton clay was excavation and the pad foundations were established on tuff. To prevent the surface water from accessing the foundation, agricultural drains were designed to serve as cut-off around the structure. Excavation for a four storey residential flat at Umoja Estate also encountered a lot of groundwater at 0.9 m depth. The water quickly flowed into the excavations and had to be pumped out as the pad foundations were cast in low water content mixes.

4.2.1.4 Ngara and Ruaraka Areas

Investigations at GSU footbridge, NACECE building at KIE and Utalii Hotel encountered clays at the surface followed by yellow tuff as shown in Figure 4.42. Crushing tests show that the allowable bearing capacity of the yellow tuff ranges from 273 to 955 kN/m² while that of grey tuff ranges between 682 and 1363 kN/m². Mathare Nyayo Hospital ward site (F25) indicates that the depths suitable for shallow footings (1.0-2.1 m) comprise of materials with fairly low bulk densities (12.5-13.5 kN/m²) and high moisture contents (average of 44.8%). Some samples from Mathare 4A Slum plotted as silty clays of medium to high plasticity (Appendix 6 (d)) and grading curves indicate that the soil can be classified as silty sand (Table 4.7).

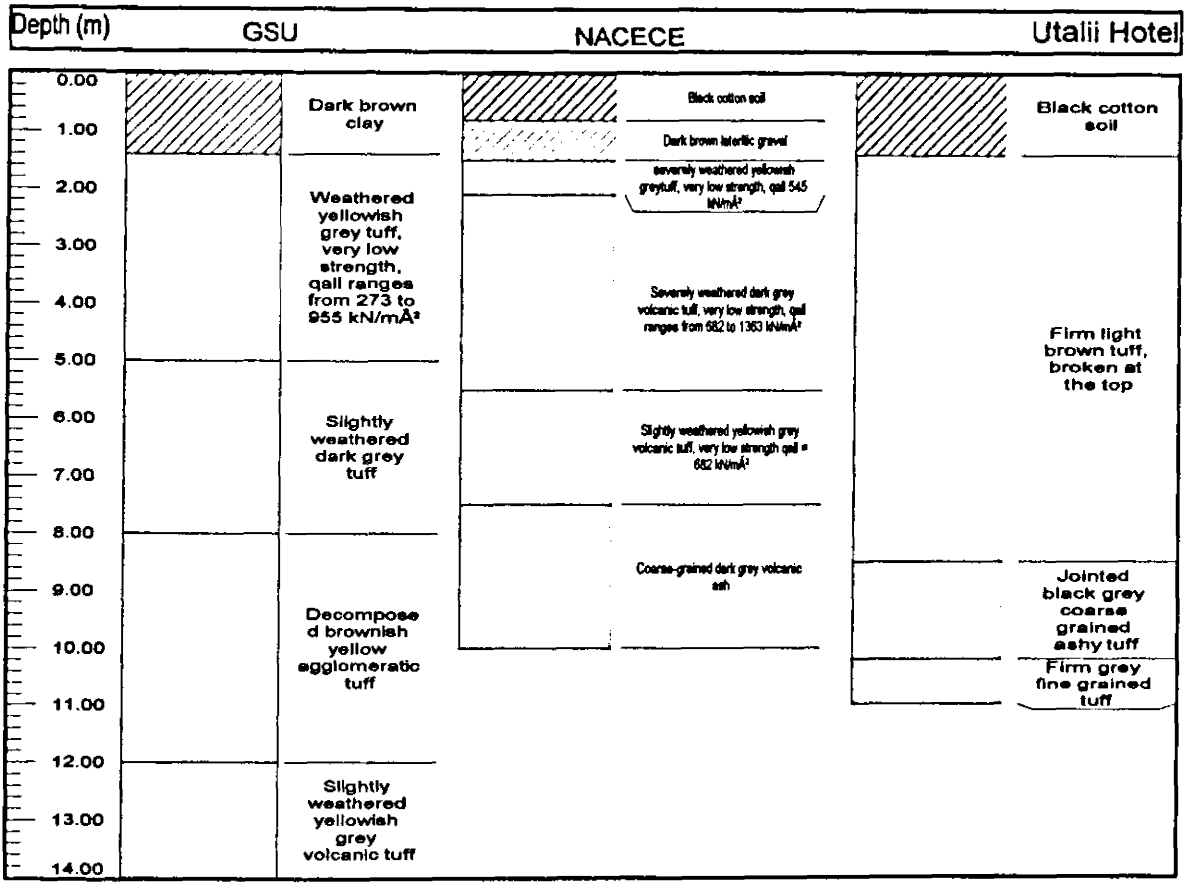

Figure 4.42: Subsurface profiles for Ngara-Ruaraka Area

4.2.1.5 Muthaiga, Garden Estate, Kasarani areas

i) Muthaiga

Failure investigation into the cause of serious cracking of a main house and guest house at Muthaiga Estate (F24) indicated that the subsoil consists of red clayey silt of a very low density and that the soil is very sensitive to settlements. It particularly has high additional settlement when flooded under load. It was concluded that the cracking was caused by uneven settlement of different parts of the buildings. The outer walls were subject to additional settlements due to flooding of the foundation base caused by very shallow foundations for the guest house and no apron around the buildings. The water sprayed on the flower beds and rain water easily entered under the foundations. For the guest house, one trial hole revealed that there was no concrete footing and reinforcement could be absent. The absence of reinforced beam or strip to give the necessary stiffness to take up uneven settlements aggravated the situation. The following were recommended: put apron around the houses; avoid flower beds, use pots; check and repair any leakages and lead all water away.

ii) Garden Estate

A November 1989 failure investigation of a house at Garden Estate (F26) reports that cracks were most prominent in the wing of the house where bedrooms above the garage basement are located more so at the corner. The floor in the garage had bulged indicating the presence of swelling clay under the floor. In addition to the major cracks, tiny cracks of less than 2 mm width existed in the other parts of the wing. Four trial pits excavated to a maximum depth of 4 m gave an opportunity for an inspection of the foundation. The pits revealed the existence of a gap between the foundation and the soil below. The gap was about 20-30 mm high at the severely cracked point and reduced gradually until it disappeared. The point where the gap disappeared coincided with the end of the major crack on the wall. It was recommended that the house be underpinned and that the new foundations are placed at 5.5 m depth below the ground by use of concrete piers. In the mean time, the garage floor would be supported by inclined struts. It was also recommended that repair of the cracks is not carried out until a few months after the underpinning is accomplished since minor movements could occur during this period. The house was underpinned after eight months.

During this study, four trial pits were dug at different parts of the building. Under a 0.5 m thick layer of black cotton soil was found brown slightly lateritic, very plastic stiff fissured clay extending to the full depth of the test pits. A sounding further down to 4.8 m indicated that the clay continued at least to this depth; the depth to rock was not found. From the pits, samples were taken for determination of water content and plasticity. The summary of the results is as follows: LL = 73-96% PL = 35-42%, PI = 38-63% and LS = 18-21%. Plotting the results on a plasticity chart indicates that the soils have high to extremely high plasticity and therefore very compressible. It was concluded that the cause of cracking of the house was shrinkage of the clay below the foundation during the periods of dry weather. This conclusion is supported by the low water contents measured in the samples just below the foundation.

iii) Kasarani Area

Kasarani Sports Complex has the highest concentration of geotechnical boreholes: 27 at Aquatic Stadium (F27), 25 at Sports Complex (F28), 24 at Athletics' Hostel (F29), 8 at Gymnasium (F30) and 17 at Athletic Stadium (F31). All the boreholes were drilled in a grid pattern. The reason for this extensive investigation was due to reports of distress in shallow foundations structures within the vicinity. The Athletics Hostel subsurface profile indicates that the site is covered with black cotton soil, red soil and brown clay. The depth of the soft material varies from 0.25 m to 2.5 m. In triaxial tests, the friction angle of the soil varies from 35° to 49° while the cohesion values vary from 2 kN/m^2 to 15 kN/m^2 as shown on Table 4.8. Underneath the soft overburden is yellow or grey tuff with increasing strength with depth. A maximum allowable bearing pressure of 400 kN/m^2 was recommended for the tuff.

At the Athletics stadium, black cotton clay was found to extend to depths of 3.0 m below the ground surface. Underlying the cotton soil was found a grey tuff mainly composed of fine grained water-laid aggregates cemented in the tuff matrix. It was recommended that the area be avoided or the black cotton soil be wasted away.

4.2.1.6 Zimmerman and Kahawa West estates

This residential area was zoned for single dwelling in the original plan but because of unplanned construction, up to ten houses can now be found on one plot. Most of the existing foundations are about 1.2 m deep and rest on laterite. There is no sewerage system in the area so every plot has its own septic tank/ cesspit. Due to large amounts of sewerage and wastewater discharged, there is leaching to the surrounding ground making construction sites for new buildings very wet. The longer term concern is what will happen when the murram becomes wetted to greater depths.

Table 4.8: The shear strength parameter ranges for some sites

Site	Cohesion range (kN/m²)	Friction angle range (°)
Kasarani	2-15	35-49
Athi River	10-330	10-46
Kileleshwa	14-80	6-39.5
Ngei	18-50	28-42
Onyonka	26-46	5-34
Kibera	42-95	6-28
Kyuna	0-100	4-44
Industrial Area	32-49	6.4-26
South C	17-120	11-35
City Centre	16-20	35-42

4.2.1.7 Community-Upper Hill Area

The subsurface formations at the Upper Hill area comprise of overburden soils and a thin layer of Kerichwa Valley Series tuffs underlain by Nairobi Trachyte as shown in Figure 4.43.

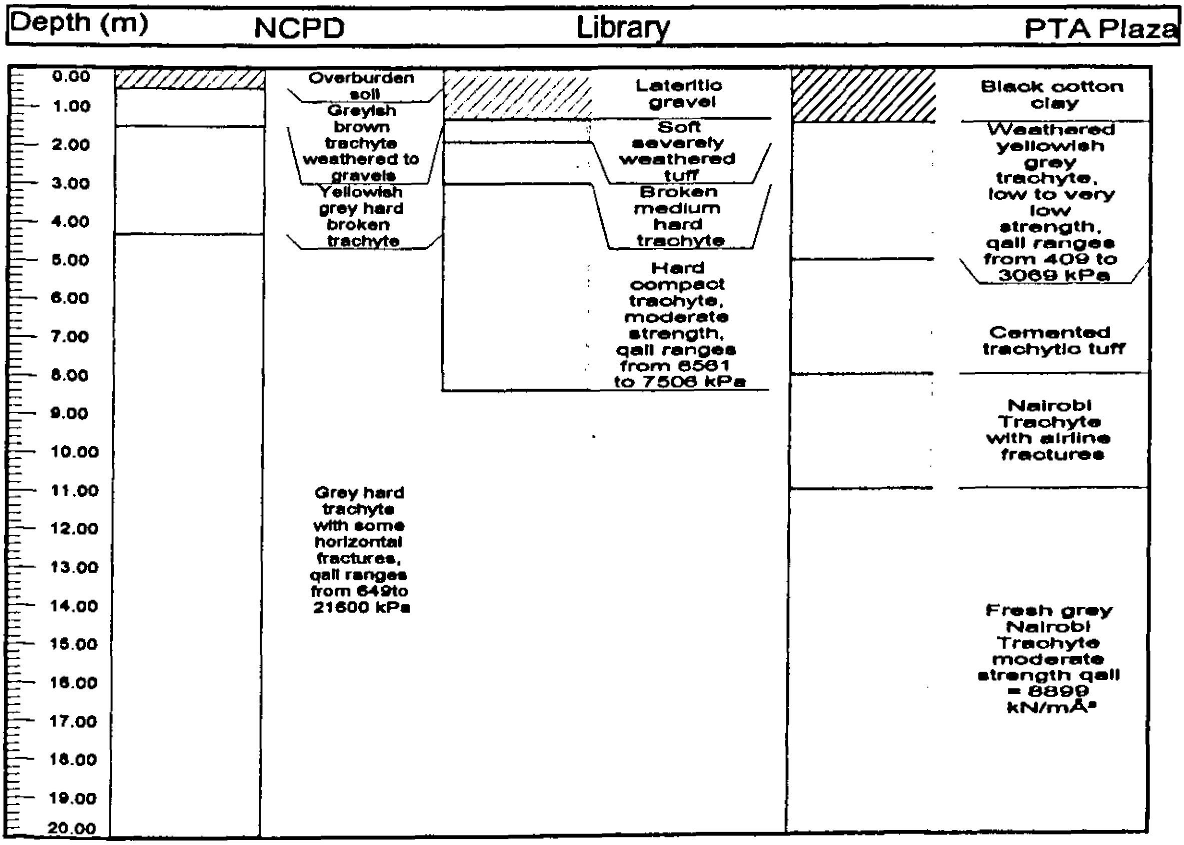

Figure 4.43: Subsurface profiles at Community-Upper Hill Area

At Ardhi House in Community area (F32), subgrade materials are poor with regard to load carrying capacity. The hard and solid trachyte found in the vicinity suddenly and surprisingly changes and exhibits high degree of decomposition at this site. The allowable bearing capacity of the trachyte was found to be in the range of 400- 3768 kN/m². This sudden change created difficulties in the design of foundations. The other office sites in the vicinity e.g. Works Office have a very shallow overburden at 0.3-1.8 m followed by fractured trachyte that makes distinct noise when hit by a geological hammer.

4.2.1.8 Westlands Area

i) Brookside Drive

A site at Brookside Drive in Westlands (F33) presented challenging design conditions for a single storey residential house. The site is within a valley covered by 2 m deep black cotton soil that may have been a spring in the past. To prevent future expansion of the black cotton soil, top soil was excavated and the site was inundated for several days to attain the maximum moisture content. The pad foundations for supporting columns were taken beyond the black cotton soil to rest on weathered rock. The sides of excavation were protected with timber shoring to avoid failure. To provide a working platform on the soaked soil and to maintain the moisture within it, a 100 mm layer of red soil was placed and covered with polythene. Spacer blocks were placed on the polythene after which the reinforced ground floor slab was cast; the loads were supported on ground beams. Polythene membranes were provided on the sides to prevent moisture migration.

ii) Westlands Motors

In 1975, Ministry of Roads and Public Works (MR&PW) was requested to carry out geotechnical site investigation for Westlands Motors (F34) along Uhuru Highway. The investigation encountered variable subsoil and concluded that a subsurface gulley, possibly an old river course may be running through the site and discharging its water to Nairobi River. Dark grey to brown silty clays of high plasticity were recovered from the test holes and groundwater table was encountered at the lower elevation. On the plasticity chart, the soils plot as silt and clay of low to medium plasticity (Appendix 6 (b)). Grading curves show that the soils can be classified as sandy silty clay. The recommendations for this site were: construction of French drains along the perimeter of the site to a depth of 2.5 m to keep down the groundwater table, removal of all organic silt from site and provision of joints in pavement slab to prevent damage caused by uneven heave. These recommendations were in accordance with the research findings by Burland *et al.* (1972).

iii) **Rhapta Road**

Construction of a four storey block of flats along Rhapta Road (F35) encountered sloping rock beneath a layer of black cotton soil and laterite. Groundwater at the site flowed along the bedding plane towards the river. Because of the slope, one side of the building could not afford a parking as the Nairobi Trachyte proved so hard to excavate. Loads on this side were supported on strip footing taken to 1.5 m depth and the ground floor was suspended above the ground. An underground tank was built to collect groundwater for irrigation and flushing.

4.2.1.9 Kangemi, Runda, Kyuna and Lower Kabete areas

i) **Kyuna and Kangemi Estates**

Geotechnical investigation for construction of Kyuna Estate (F37) in 1973 described the red soil at the area as sandy silty clay of high plasticity. On the Casagrande plasticity chart (Appendix 6 (b)), the soils are uniform and have medium to high plasticity while on the grading curves the soils can be classified as silty clays. Double oedometer test results indicate that the soil is liable to settlement under natural moisture content and collapses when flooded. Unconsolidated undrained triaxial shear test carried out on various samples obtained cohesion values ranging between 40 and 100 kN/m^2 and friction values ranging from 23° to 29°. To avoid cracking of houses due to uneven settlement the following recommendations were made: use of inverted T-beam foundation on red soil; maximum allowable bearing pressures of 150 kN/m^2 and provision of good surface drainage.

Similar conditions were encountered at the Kangemi United Club (F38) and National Soils Laboratories (F39) and the same design measures were proposed. In addition, settlement monitoring after construction was recommended for the F38 and F39 sites. This was after non-flooded oedometer tests indicated settlement ranges of 56-108 mm when the soils are loaded with 200 kN/m^2 and 500 kN/m^2. Under flooded conditions, the soils were expected to experience 84 mm settlement under 200 kN/m^2 and 123 mm under 500 kN/m^2 pressures.

Distress due to collapse settlement of structures supported on Kabete Red Clay has been reported in the neighbourhood of Kabete Armed Forces field (F40). The site proposed for construction of a hospital was investigated by means of trial pits dug to a maximum depth of 3 m. All the trial pits encountered the red clay. Physically the clay was described as being of high plasticity with thixotropic characteristics. In the natural state it looks dark brownish red, open textured and granular like silt. On the plasticity chart the soils plot in the medium plasticity range (Appendix 6 (d)). Collapse test results indicate hydrocollapse strain values ranging from 0.08 to 0.84 when flooded at 50 kN/m² and 150 kN/m² (Table 4.6). According to the Coduto (2001) criteria, the soils classify in the range of no trouble to very severe trouble.

Construction report for 120 single-storey residential units for the National Social Security Fund at Mountain View Estate in Kangemi (F41) indicates that the site is covered by red soil of appreciable bearing capacity. Due to the undulating nature of the ground, foundations for some structures were constructed on cut and fill ground. Some eleven plots could not be developed because they were in the vicinity of a spring of water that joins Nairobi River.

ii) Nyari Estate

Construction of a one-storey residential house in Nyari Estate (F42) encountered groundwater flow problems and black cotton soils up to 2 m depth. To avoid the problems associated with black cotton clay, reinforced concrete piles driven to a depth of 2.5 m below ground level were designed for use. A suspended ground beam supporting load bearing walls was provided to carry all the loads. To minimize pressure on the slab and the settlement associated with thick hardcore, 300 mm of black cotton soil was excavated and replaced with red soil. After completion, this type of foundation was found to be slightly more expensive compared to the alternative raft foundation design with inverted beams. The soil properties at the nearby Kitisuru NSSF estate are uniform clayey silts of medium plasticity (Appendix 6 (c)). On the grading curves the soils can be classified as clayey sandy silt.

iii) Lower Kabete Area

A single-storey masonry structure near Red Hill Road (F43) exhibited distress at one corner of the building. Geotechnical and structural investigations of the site and structure were undertaken. A trial pit excavated beyond the foundation base indicated the presence of termite sinkholes at the distressed corner but none along the wall. The evaluation also determined that the structure had undergone edge curl at the distressed corner; consequently the foundation base was 100 mm lower than the expected grade. It was concluded that foundation section above the sinkhole was settling resulting in higher levels of foundation distortion along the area of influence. The sinkhole was probably present for a long time prior to the damage. Growth of the sinkhole was accelerated by rainwater percolating from a parking lot beside the building into the strata beneath the structure. The downward migration of the soil into underlying openings and the formation and collapse of the cavities were accelerated by increase in the velocity of movement of water and induced recharge. It was recommended that compaction grouting be done to raise the footing and proper drainage and grading around building be carried out.

4.2.1.10 Madaraka, Nairobi West and Nairobi South Areas

i) Madaraka Estate

Madaraka Estate exhibits very sensitive and variable subsoil conditions. Construction of a three-storey office block for Bible Society of Kenya (F44) indicated the presence of a layer of black cotton soil sitting on fractured rock with groundwater seeping through the fractured rock at high velocity. The structure was designed to be supported on pad footings combined with the basement slab at 3 m depth. Due to pressure from groundwater within the fractured rock, seepage occurred through the slab and hampered the work progress. Chemical treatment done on the surface of the slab and foundation completely controlled the seepage. A ten year guarantee was given by the manufacturer in case the seepage recurred. French drains were provided around the structure to allow groundwater to flow freely below the foundation to join the river. The system worked and no problem has been reported since.

Several structures that were constructed in the late 1970s within the compound of Madaraka City Council Primary School (F49) were not in use at the time of this study. A failure investigation report done on 15 June 1980 indicates that an existing two storey main building developed big cracks (2 cm wide) on the wall and some corner. Neighbouring wings also had some smaller cracks. Three test pits were dug to depths of between 2.1-3.5 m to expose the foundations of the main building. The materials underneath the foundations were found to comprise of a layer of yellowish-brown silty clay with bulk densities ranging from 15-16 kN/m^3 and water content between 38 and 43%. This main building has undergone several repairs but it has not been restored to continuous use. One of these neighbouring buildings that is probably supported at a shallower depth is shown in Figure 4.44. Though there is good paving around the structure, it has visibly sunk into the ground. A relatively new structure within the compound that is supported on ground beam has not settled as much but the ground around it has, causing separation between the ground beam and subsoil.

Figure 4.44: Settled building at Madaraka Primary School

Geotechnical site investigation carried out in 1990 for construction of the workshop within the Madaraka Primary School compound was done to a depth of 3.6 m. It was found that a black clay layer exists from the ground surface to 1.5 m followed by yellowish-brown silty clay with the following properties: natural moisture content 30-40%; unit weight 15.1-16.5 kN/m^3; Linear Shrinkage 15-21%; PI 45-54% and LL 69-85%. On the plasticity charts, the soils plot in the range of high to extremely high plasticity (Appendix 6 (d)). The clay was also found to be collapsible with potential hydrocollapse index varying between 0.142- 0.198 meaning that the soil can be of severe trouble to structures. It was recommended that the black cotton soil be excavated and wasted, the clayey silt be excavated to 1.5 m beyond the cotton soil and recompacted in layers of 300 mm thickness before setting the foundations. It was also recommended that: the foundations be made stiff to avoid problems associated with differential settlement; an open space be left beneath the foundation to allow for possible heaving of soil if the foundation is supported on the black cotton soil; apron be provided around the building; an effectively sealed expansion joint be provided between the apron and the wall; and, all the surface drainage be lead away. The workshop constructed at the site failed as shown in Figure 4.45.

Figure 4.45: Distressed workshop at Madaraka Primary School

During the period of this research, four building projects were going on concurrently at Strathmore University within Madaraka Estate. Various subsoil problems were encountered at the construction sites. The most prominent subsurface feature is the sloping underground strata in the vicinity of a river. The depth of black cotton soil in the compound was found to be in the range of 2 m to 6 m. This reverse gradient caused flow of water along the bedding thus softening the subsoil and flooding construction excavations. French drains were constructed to control groundwater flow while sump pumping was continuously done. Foundations at three sites were designed to utilize U-Boot technology that makes slabs that are lighter than the conventional slabs thus reducing the prospect of excessive settlement. The fourth construction site that is located at a site with 6 m deep soft clay was designed as a raft foundation with steel columns to mitigate total and differential settlement.

ii) Kongoni City Council Primary School

Within Nairobi South area, subsurface conditions at Kongoni City Council Primary School (F50) are remarkable. The profile consists of approximately 3 m thick very soft to soft clay overlying laterite with the water table at 0.5 m below ground level. The foundation system employed for a distressed workshop within the compound was a soil-supported, monolithically-cast, reinforced concrete mat foundation assumed to be floating type. Damaging building settlement occurred within the first few years after completion of construction; cracks developed on the walls and masonry columns, soon ceiling was falling on the pupils (Figure 4.46).

Investigations were conducted to identify the cause of the building settlement and develop remedial foundation design. The structure was found to be beyond repair and condemned in 1996. A visit to the site during this study also attributed the quick failure to existence of two very large trees 2 m away from the pavement. A profile exposed on the sides of an excavation for construction of an ablution block within the compound comprised of clayey silt up to 3 m depth. The problems encountered during foundation placement were large inflows of groundwater combined with leakage from surface mains making the excavation walls to collapse three times.

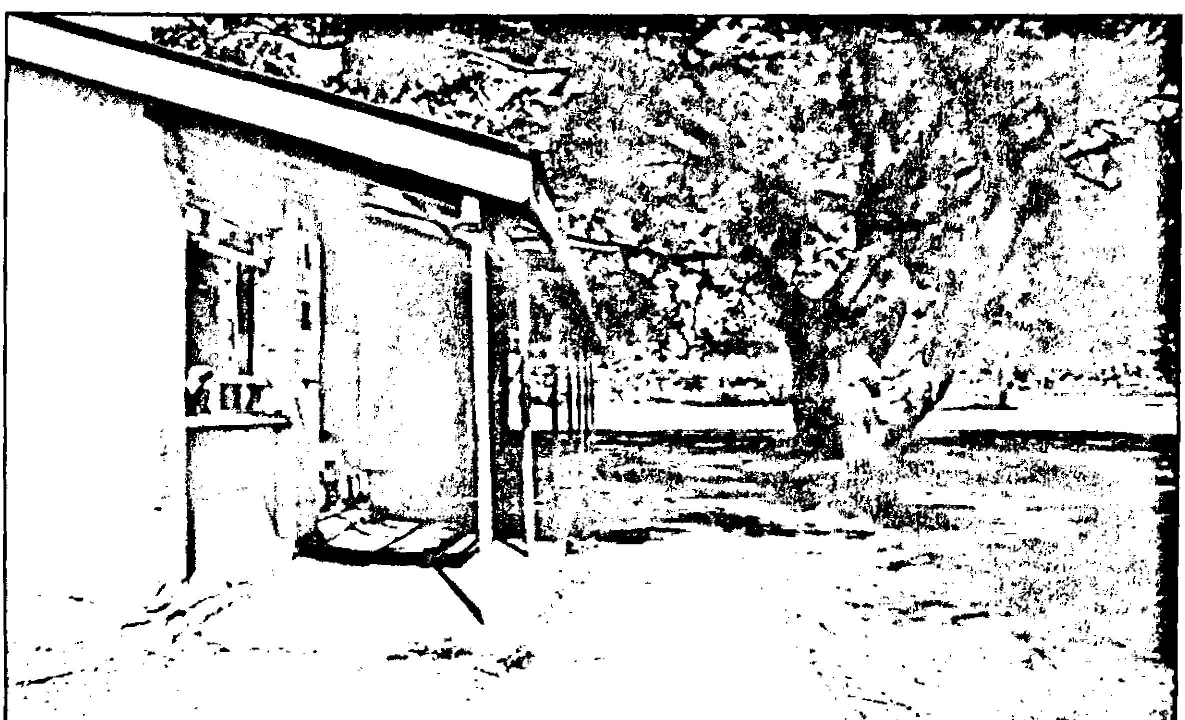

Figure 4.46: A failed workshop at Kongoni Primary School.

iii) **Akila Estate**

Site investigation for the construction of maisonettes at Akila Phase II Estate (F51) encountered 2 m deep black cotton soil followed by laterites with water flowing at the interface. The Phase I maisonettes founded on a thin layer of this black cotton soil were damaged causing the developer to take the contractor to court. To build the Phase II maisonettes, all the black cotton soil was excavated. Polythene was laid on the underlying soil after which mass concrete was placed to prevent undue swelling and subsequent reconsolidation of the soil. The concrete was placed at 45° to increase bearing area. Construction of the pads to support columns proceeded. Moisture barriers were placed and paving was developed all round the substructure. Drainage was established to lead the water away from the site into a river nearby. The project also repaired perimeter walls that had started collapsing by use of anchoring blocks and cables.

iv) **Belle Vue Estate**

Failure investigation of Belle Vue Estate maisonettes (F52) was done through test pits. Visual examination of the test pits revealed that the two-storey maisonettes were supported on 110 mm thick slab-on-grade foundations. The slab was not thickened at the edges under the load bearing walls. The soil profiles also indicated that the site had been levelled by cut and fill operation and that the depth of fill was variable, ranging from a few centimetres to over two metres. The fill consisted of the local dark grey/brown highly plastic clay with cobbles of tuff varying in size from a few millimetres to over 300 mm. The ground floor slab was cast on the levelled ground. The damp-proofing viscreen under this slab was visible near the external walls of most of the houses, thus the slab is practically above the ground. The test pits also exposed random large voids and vegetable matter at some places. It was concluded that the causes of distress were differential settlement on sloped fill and consolidation of organic soil and plastic clay.

4.2.1.11 Langata –Karen areas

i) **Ngei Estate**

Geotechnical investigation for construction of Ngei Estate (F53) was carried by inspection of trial holes and plate loading tests at the base of the excavations. The subsoil that consists of lateritic clay at some sites failed under instant static load of 60-80 kN/m². A sample of the clay was collected and tested in a wet condition. After 24 hours of soaking, the material crumbled at very low unconfined compressive stress (8-12 kN/m²). Direct shear tests obtained cohesion values of 16-40 kN/m² and friction values of 28°-42° (Table 4.8). The following recommendations were made: placement of foundations below the lateritic clay; use of stiff concrete ring beam for supporting loads; provision of hardcore and polythene sheeting under floor slab and development of apron around the structure.

By 1979, several Ngei Estate house owners such as the one in Figure 4.47 were complaining of ground separation, cracks on floors, walls, beams and masonry. In order to determine the cause of distress and to obtain quantitative data, trial pits were dug to expose the foundations. Physically the foundation soil was described as dark grey clay with very fine clay minerals, possibly bentonitic. Laboratory tests on a sample taken from the house in Figure 4.47 yielded very unusual results as follows: LL 115%, PL 35%, PI 80%, linear shrinkage 19%, clay content 82% and silt content 14%. On the plasticity chart, the soil classified as clay of extremely high plasticity.

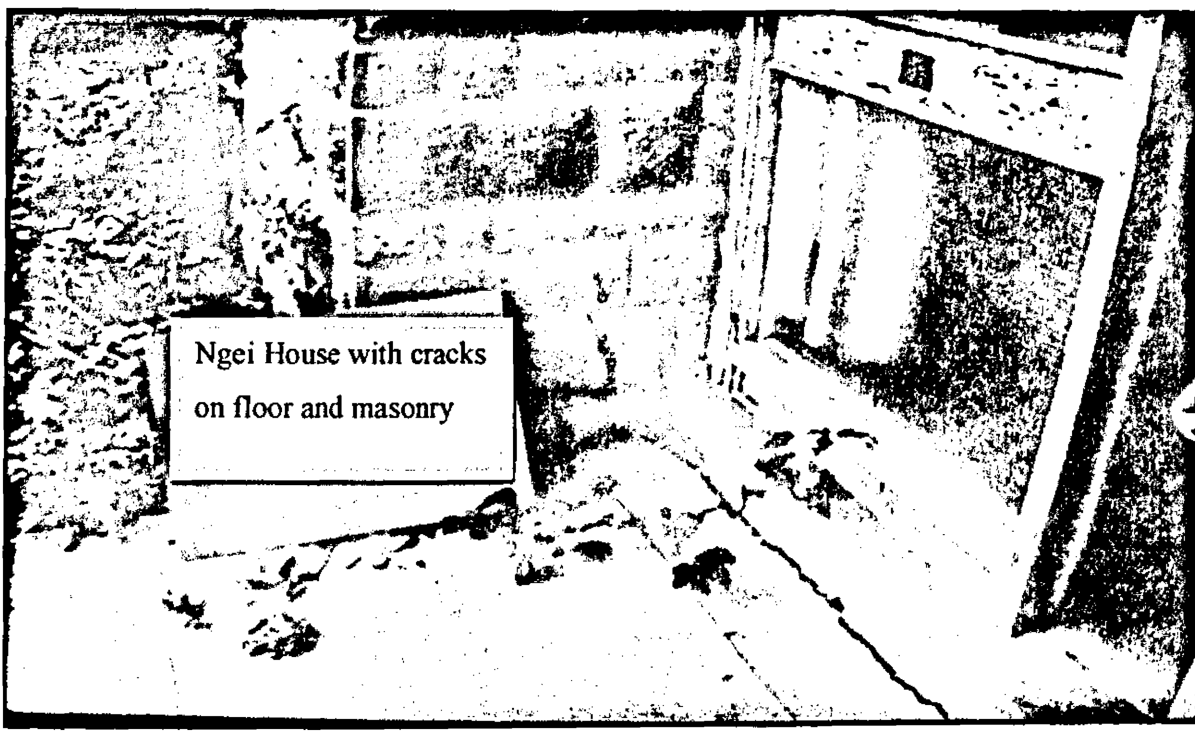

Figure 4.47: Distressed house with cracks in the floor and masonry
(1979 file photo courtesy of MR&PW)

Test pits at the house shown in Figure 4.48 revealed that block courses at the cracked walls were higher than at other locations suggesting that they may have lifted up. Pictures taken on the interior show that the middle of the building remained virtually stationary while the sides lifted up. This caused development of depressions in the central part of the house. If it is assumed that the middle of the structure did not move, upward movement of about 80 mm may have taken place at the ends of the building in five years. It was also observed that there was no proper drainage at the distressed houses.

In January 1985, another investigation into the cause of cracking in some other houses in Ngei Estate was carried out. The soil supporting the foundations was described as dark grey clay. The swell pressures of the clay were found to be in the range of 156.4 - 206 kN/m². Given that the design pressures adopted on this soil type are in the range of 100-180 kN/m², the swell pressures were enough to lift the foundations upwards.

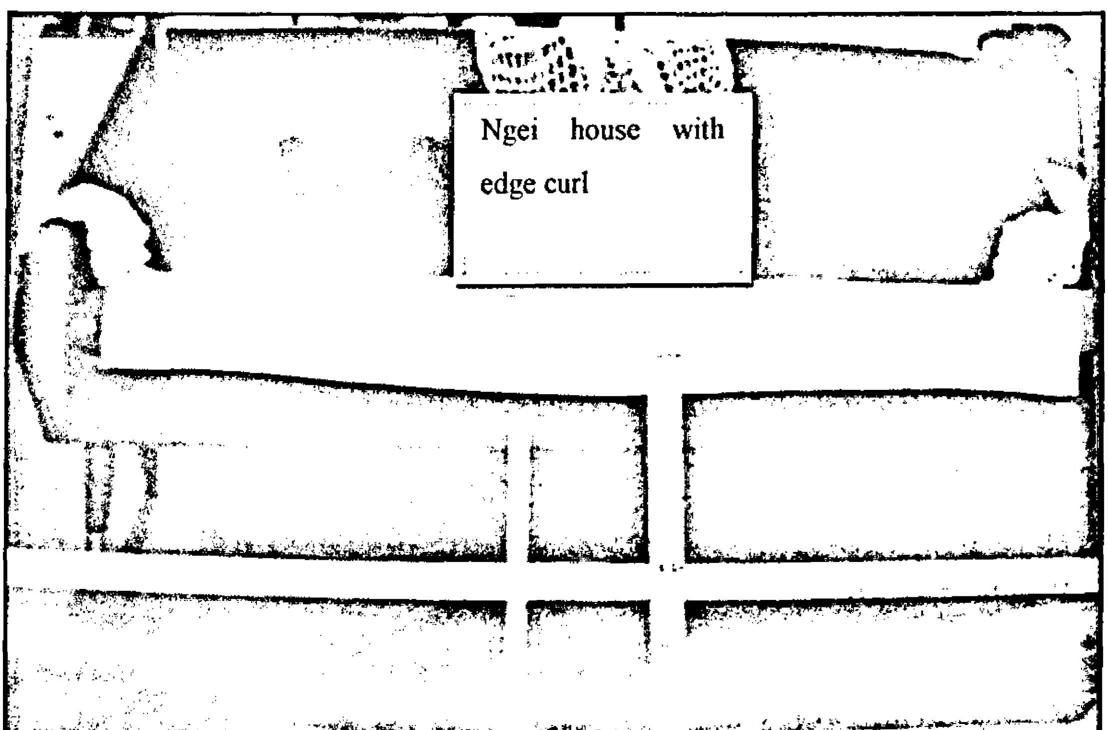

Figure 4.48: Distressed house with depression at the centre

(1979 file photo courtesy of MR&PW)

ii) **Onyonka Estate**

Results of geotechnical investigation for Onyonka Estate ((F54) in 1980 shaded more light to the cause of distress in the Ngei Estate structures. Excavation for foundation placement of the National Housing Corporation (NHC) houses encountered varying subsoil conditions. The 26-trial pit investigation was mainly centred in areas where founding levels looked suspicious in composition. So this was a confirmatory site investigation since foundation construction had already begun. The predominant soil formations encountered in the trial pits are layers of black cotton soil and dark brown highly plastic clay. The overburden extends from the ground surface to an average depth of 2.8-3.0 m where laterites of varying thickness and hardness are found. Within the soft overburden, occasionally encountered are lateritic clays with much semblance with laterites which can be easily misleading on bearing capacity. They are just clays with scattered fine gravel and some iron oxides of clay and gravel. These lateritic clays are very tricky for the foundation designer since they appear like laterite and would consequently be assumed to have same bearing capacity as laterite, an assumption that would be absolutely wrong.

On the plasticity charts, the Onyonka Estate soils plotted at medium to high plasticity range (Appendix 6 (a)). Consolidation settlements tests indicate that parts of the overburden are highly compressible, and most of the soils have swelling characteristics when wetted. Direct shear tests indicate that the internal friction angle is in the order of 28°, triaxial shear parameters indicate low angles of internal friction (average 10°). Thus the average angle of internal friction based on triaxial shear and direct shear is in the order of 19°. This angle being so is a manifestation of the high clay content. Grading curves indicate variations in gravel content across the area. The distress in some building floors and walls in Ngei Estate Phase II is believed to be due to the assumption that the lateritic clay has similar properties with laterites. The load carrying capacity of the lateritic clays is very low and settlements are often excessive and unreliable in wet conditions whereas the lateritic gravels have considerably high bearing capacity even after soaking under water for long periods. In this region, buildings ought to be founded on excavated piers so that the column bases are isolated and the walls supported on ground beams.

iii) **Mbagathi Central Training School**

In June 1975, an investigation was carried out into the assumed settlement on either end of one of the buildings at Nairobi Mbagathi Central Training School (F55). The cracks had developed some time after completion and the damage had worsened over the succeeding years. Four trial holes were excavated to expose the foundations at the four different sides of the building. Tests on the samples showed that the subsoil is silty clay of high plasticity overlying a laterite layer. It was noted that aprons had been constructed on either side along the building but on either end, rainwater could easily enter under the building thus influencing the soil under the foundations. With a roof overhang and a properly constructed apron at the ends of the building, future damage was expected to be minimized. New building foundations on this soil were recommended to be stiff ground beam. A geological log for a water supply borehole (C-3159) within the compound describes the soil from the ground surface to 200 m depth as boulder bed (sheared material). The distress may be due to the consolidation of this faulted material.

iv) Langata Road

In 1970, a failure investigation was carried out at a house along Langata Road (F56) at Karen. The house was inspected and samples were obtained from trial holes. The investigation report indicates that the house had cracks that were mainly vertical with a few horizontal and diagonal ones. In the field, it was observed that the cracks may have resulted from differential settlement of the building. The pattern was, however, somewhat more complex than that usually associated with red soil indicating that the problem could be attributable, in part, to shrinkage occurring during dry periods. On further inquiry, the client informed the engineers that the house had previously been underpinned during which the foundations had been deepened. The cracking could also be attributed to settlement of the new foundations due to underpinning. Samples obtained at 1 m and 2 m depths were of clayey soil of high shrinkage characteristics on an otherwise well-drained site. The shrinkage limit tests carried out on two samples show that the shrinkage limit for this soil is around 26%. This was the same moisture content at the time of inspection and it was concluded that no further shrinkage could occur as the cracks were at their worst. It was recommended that the cracks be repaired as no further settlement was likely to occur.

v) Warai North Road and Miotoni Road

A house along Warai North Road (F57) suffered serious damage that necessitated an investigation. Flooded and non-flooded oedometer tests at different loading conditions were carried out on undisturbed samples taken from underneath the foundation. The three samples showed collapse of the soil structure when saturated with water. The reasons for the increase in moisture content in the foundation soil were identified as leakage from the sewer system and access of the rain water into the foundation. When the position of this site was plotted on the geological map, it plotted at the edge of a known fault (Figure 4.36). Along Miotoni Road (F58), construction of maisonettes encountered deep plastic alluvial deposits. The foundations could not be set at the normal 1.2- 1.5 m depth that was supporting most of the existing structures within the vicinity. A water supply borehole within the vicinity indicated that the soft overburden extends to 12 m below the ground surface. The pad foundations were set at 5 m depth and broken at intermediate sections by beams.

vi) Karen Shopping Centre

Off Karen Road (F59), a successful project was completed within a swampy area that had been sold at throw away price. Geotechnical investigations carried out at the site indicated that the soft soils extend to a depth of 8.5 m. Groundwater was encountered at 20 cm below the ground surface. The method of construction consisted 8.5 m deep point-bearing pile foundation and a suspended ground floor slab. The ground below the structure was left intact. The whole compound was concreted to ensure that no surface could pass through the concrete to maintain constant moisture content around the house. A layer of soil was laid on the concrete for gardening. Surface water drains were developed around the compound to tap the water flowing from above the concrete and lead it away. Groundwater within the compound was also allowed to flow undisturbed to join the river. At Karen Police Station (F60) brown highly plastic soils were also encountered and continuous ring beam foundation and use of polythene sheeting to prevent soil moisture change were recommended.

vii) Co-operative College

Preliminary site investigation for Co-operative College (F61) observed a two storey building nearby that had been constructed three years earlier. A resident in this building informed the investigation team that during construction, black cotton soil was encountered up to a depth of 3 m below ground level. The paving slabs on the periphery of this building appeared cracked and distorted indicating that some heave had occurred within the subsoil. However no cracks were visible on the outside walls of the building.

4.2.1.12 Bernhard and Lavington Estates

i) Bernhard Estate

In 1973 an investigation was carried out into the cause of cracking of a house at Bernhard Estate (F62). On inspection, it was observed that the exterior walls of the house were in good condition while the partition walls had serious cracks. It was concluded that the defects were of local nature and had no connection with the general soil conditions. The settlement was attributed to pressure from a thick layer of hardcore placed during construction. The internal walls settled around the floors and domed pulling the floors with them. This caused development of cracks that were wider at the upper parts of the walls indicating local settlement. When plotted on the geological map, this site falls nearby an area indicated as consisting of swamp and alluvial soils.

ii) Lavington Estate

Construction of a two storey house opposite Braeside School at Lavington Estate (F63) encountered deep black cotton soil and high groundwater level. Because of subsoil conditions, uplift and differential settlement/ heave were expected. The construction comprised of heavily reinforced and waterproofed raft foundation supporting columns and load bearing walls. The raft was also made heavy enough to overcome uplift and swell pressures. It was not possible to obtain information from the vicinity to indicate if any of the structures were undergoing distress or if deep foundations had been adopted in their construction.

Trial pits done to investigate a site for construction of a three storey residential cum commercial house at Lavington Shopping Centre encountered 1.5 m thick layer of black cotton soil underlain by laterites and groundwater at 1 m. Reports from sites within the vicinity indicated existence of an underground river. The construction excavation also encountered old pits. To eliminate the water from the excavation, trenching was done and the water was pumped out. Before foundation placement, the consultant noted that the excavation bottom at 1.5 m was too weak and requested the contactor to excavate a pit beyond base in order determine the reason behind the weakening of soil. The exercise revealed that the gravel layer that had been designed for foundation support was very thin (600 mm) and underlain by weak decomposed rock extending to 6 m. This necessitated use of 6.0 m long in situ point bearing concrete piles for structural support. No distress has been observed since the structure was completed more than a decade ago. These weak conditions were however not noted in the adjoining plots because the foundations are shallower. The structures could therefore be liable to settlement.

A single storey residential house at a certain site within Lavington Estate experienced distress. Trial pits dug to expose foundations revealed that the site is covered by very deep soft soils. The pad foundations for the main building were resting at 6 m below ground level. The house owner decided to make an extension that was sharing a wall with the main house after some time. The foundation of the extension was not taken down to the level of the foundation of the main house. Resting on the soft subsoil, the extension settled and pulled the main house with it. This settlement induced cracks mainly on the shared wall; the owner could swear feeling the house quake every night. To fix the problem, the foundations of the extension were underpinned to the level adopted for the main house before retrofitting and repainting.

An investigation into the cause of distress in one of the buildings along Gitanga Road (F65) revealed that the distress in the structure was due to severe underground erosion. It was proposed that the affected structure be demolished because the prevailing conditions at the site would not allow for remediation. A core log for the well (C-10615) within the compound indicates that this decomposed and saturated stratum extends up to 28 m below the natural ground level.

4.2.1.13 Kileleshwa, Riverside and Milimani Areas

i) Kileleshwa Estate

Site investigation for construction of a four storey block of flats at Kileleshwa (F66) revealed that the site has 7 m deep black cotton soil and groundwater flow problems. Pad foundations taken beyond the black cotton soil were designed for use. The ground floor was suspended above the ground. Because the columns were too high, they were tied at intermediate sections to avoid buckling. A waterproofed underground concrete tank was built for storing the water before it is pumped out. A nearby construction site revealed the presence of 6 m thick non-uniform fill underlying the generally flat surface.

Investigation for the civil servant scheme houses in Kileleshwa obtained the following results: on the grading curves, the soils classify as clayey silts (Table 4.7); on plasticity charts the soils are of medium plasticity (Appendix 6 (a)); non-flooded settlement values range between 32-44 mm under 200 kN/m² load and 42-65 mm under 500 kN/m² load while flooded settlement values range from 42-84 mm under 200 kN/m² load and 68-140 mm under 500 kN/m² load (Table 4.5). Direct and triaxial shear tests yielded friction values of 29°-39.5° and cohesion values of 10-80 kN/m² (Table 4.8).

ii) **Riverside Drive**

Construction of six-storey residential blocks of flats along Riverside Drive (F67) encountered 3.0 m deep red soils overlying sloping rock with groundwater flow along the bedding. Because of the sloping nature of the rock, columns bases were cut into rock and tied together to avoid sliding while all basement walls were designed as concrete diaphragm walls. Chemical analysis results of the groundwater showed that it contains high amounts of fluoride and chloride and hence could not be used for construction. Because the amount of groundwater flowing was large, a plant was constructed to treat it for use within the buildings.

iii) **Lenana Road**

Construction of four-storey blocks of offices at Jumuia Place (F68) along Lenana Road encountered notable subsoil conditions. The soil profile at the site consists of 1.5 m deep black cotton soil underlain by rock. During construction, it was found that the rock at 1.5 m depth was too hard to be excavated for a basement for one of the blocks and water was flowing from the rock at high velocity after a rainy season. To avoid project delay, the foundation was placed at 1.5 m depth and the building was raised above the ground to accommodate parking. For the second block that was located on the lower elevation, the soil was excavated to accommodate full parking. The sloping ground was retained by masonry wall and levelled so that no slope is apparent within the compound.

4.2.1.14 Kibera, Woodley and Ngong Road Areas

i) **Jamhuri Estate**

Trial pits dug near Guadalupe at Jamhuri Estate (F69) indicated that weak plastic subsoil extends up to 9 m. With such weak subsoil, if the three-storey block of flats was supported on shallow foundations it could undergo settlement beyond tolerable limits. Pile foundations were found to be the most suitable option but could not be used because of the limited project finances. To safely use pad foundations, the site was excavated up to 2.5 m depth and the excavated material was wasted. The excavation was then filled with suitable imported material compacted in thin layers of 300 mm and the foundations were set at 1.2 m below natural ground surface. The ground floor was suspended well above the ground surface; all the loads were supported on ground beam. A 2 m allowance was left from the perimeter wall in case a problem occurred and necessitated remedial measures. That was late 1990s and no problem has been reported to date and that means the method worked well.

ii) **Adams Arcade**

During construction of maisonettes along Ngong Road at Adams Arcade (F70), excavations for pad footings were done to about 6 m without reaching rock. The upper 0.9 m of the soil profile consisted of black cotton soil and further down was found very stiff and fissured brown highly plastic clay. In the deepest excavations that were about 6.5 m water had been encountered. An oedometer test carried out on the samples yielded settlement values of 40 mm under 200 kN/m² load and 49 mm under 500 kN/m² load (Table 4.5). No records are provided on how this problem was dealt with.

iii) **Kibera**

Site investigation for Sonning Road in Kibera recovered very variable soils. On the plasticity chart they classified from low to high plasticity (Appendix 6 (b)). Settlement values ranged from 9-35 mm under 200 kN/m² pressures and 13-45 mm under 500 kN/m² pressures at natural moisture conditions (Table 4.5). Site and Service scheme investigation in Kibera obtained the following results: on the grading curves the soils classify as sandy silty clays; on the plasticity chart they plot at medium to high plasticity (Appendix 6 (b)); shear tests obtained friction values of 6°-28° and cohesion values of 42-95 kN/m² (Table 4.8).

4.2.2 Bearing capacities of the various geologic units

A summary of calculated bearing capacities and selected allowable bearing pressures based on test results on some geologic units from some sites is presented in Appendix 10. If the design values used in Appendix 8 and Appendix 9 are compared with the values based on test results in Appendix 10, it can be noted that most of the past foundation designs used conservative values.

Karura Trachyte, Limuru Quartz Trachyte and Kabete Trachyte are very competent rocks when fresh. The trachytes are generally fine-grained and weather to red/brown soils or plastic clays with low density and sensitive to settlement when wetted under load. In addition, the soils formed from Kabete and Karura trachytes cave in when boreholes are drilled through them and are in most cases cased from the surface up to 15 m. A system of shoring is required when excavating these soils for foundation placement especially if the excavation will remain open for some time. Kabete red clays, as the soils are commonly referred to, have caused distress in shallow foundation structures owing to their moisture sensitivity. The clays are also subject to burrows and termite sinkholes.

The Kerichwa Valley Series (KVS) that mainly consists of tuffs covers nearly one half of the Nairobi surface. Majority of the shallow foundation structures are supported on Kerichwa Valley tuffs or their weathered equivalents. The character of the Kerichwa Valley Series rocks has been investigated intensively at the City Centre and Kasarani Sports Complex. The KVS is stratified in alternating thin and strong layers that include the following starting from the ground surface: black cotton/plastic clay; laterites/fractured tuff; fine-grained yellow (yellowish grey/grey) tuff; dark grey to black tuff; and, fine-grained yellow tuff becoming agglomeratic with depth. However, the succession along Moi Avenue and Uhuru Highway at the City Centre differs slightly. At Lillian Towers, brown tuff, marine clay and coarse-grained (probably the agglomeratic tuff) tuff were recovered. The brown tuff encountered at Lillian Towers site is also found at Union Towers and at various sites along Tom Mboya Street. The brown tuff that becomes agglomeratic with depth is also found at sites along Uhuru Highway such as at Continental House. Given the depth and the state of the brown tuff, it seems probable that it is the same tuff that is reported as yellow tuff in other parts of the city. The difference in colour may be attributed to the groundwater flow since the two locations are associated with old river courses.

The KVS rocks have low to very low unconfined compressive strength (UCS) according to ISRM (1981) classification. The range of UCS values for most of the KVS tuffs is 2 to 5 MPa in which case the allowable bearing capacity (q_{all}) ranges between 500 kN/m² and 1400 kN/m². At several depths at Reinsurance Plaza at the City Centre, the tuffs crumbled while dressing. Several other sites at the City Centre indicate that the tuffs could not be sampled as a core at depths ranging between 3.7 m and 12 m. At the NSSF site in Karen a very low strength of 0.745 MPa was encountered at 7.4 to 7.7 m depth with a q_{all} of 201 kN/m². Look (2007) reports that tested igneous rock strength ranges from less than 15 MPa for welded tuffs to 800 MPa for basalt.

The black tuff is encountered only within the City Centre and at Serena Hotel site. The profile at the University of Nairobi Main Campus is as follows: 0-1.5 m black cotton clay, 1.5 m-8 m decomposed yellowish brown tuff, 8 m-18 m grey tuff followed by Nairobi Phonolite. This succession is similar to that encountered at Ngara and Ruaraka areas where yellow and grey KVS tuffs overlie the Nairobi Phonolite. The absence of black tuff at Uhuru Park and University of Nairobi Main Campus seems to support Report No. 182 at Materials Department that indicates that Moi Avenue runs along the old course of Nairobi River and other unpublished reports that indicate that Kenyatta Avenue/Valley Road run along the course of an old river, probably a fault.

Dark grey/black tuffs are very competent rocks with allowable bearing capacity of 1600 kN/m². They are encountered at 3 m to 8 m below the ground surface. The upper and lower parts of the black tuff are grey or dark grey, the middle part is black and the most competent. Design pressures for most structures resting on the black tuff are in the range of 400-800 kN/m². The reason why most structures supported on the black tuff are not designed for higher pressures is the presence of yellow tuff underneath that is often weathered to soft and plastic clay. In the presence of weathered rock or plastic clay, design bearing capacity of 400 kN/m² is used and the black tuff is considered to provide good load spreading and rafting effect. Design values greater than 400 kN/m² have been used at sites where the underlying yellow tuff is not so weathered.

Grey tuff with bands or seams of yellow tuff otherwise referred to as yellowish-grey tuff is generally more competent than the yellow tuff. Allowable bearing capacity of fresh grey tuff is in the range of 1200-3000 kN/m². Weathered grey tuff has an allowable bearing capacity range of 400-800 kN/m² while laterite formed from weathering of the grey tuff has allowable bearing capacity of 400 kN/m². However, grey tuff in low-lying areas has been found to be very weak.

Yellow agglomeratic tuffs consist of phonolite boulders poorly cemented in a matrix of yellow tuff. The agglomeratic tuffs are encountered at depths starting from 6 m and can extend up to 20

m. The allowable bearing capacity of the agglomeratic tuffs is found to be low (less than 300 kN/m²) when the matrix is tested and very high (up to 3000 kN/m²) when the boulders are tested. For instance at a depth of 8 m at KPC site in Industrial Area and 5.8 m to 11.8 m depth at NSSF Karen, very high strength values corresponding to the strength of Nairobi Phonolite are recorded. These "competent" rocks are not encountered in the adjacent boreholes indicating the tuffs are agglomeratic, and possibly the adjacent boreholes recovered the matrix part of the agglomeratic tuff. Design bearing pressures of 350-400 kN/m² have been used for structures resting on these tuffs but test results indicate that the allowable bearing pressure range is 500-800 kN/m².

Nairobi Trachyte has moderate to high unconfined compressive strength though few spots of low and very low UCS values are recorded. The strength of the Nairobi Trachyte is very low to moderate at the Hill Plaza site at Community area. At the Library site along Ngong Road, the UCS ranges from 12 000 - 27 000 kN/m², at National Council for Population Development (NCPD) site at Community it ranges from 2 000 - 80 000 kN/m². The minimum allowable bearing capacity of 409 kN/m² was encountered at a depth of 3.85 m at PTA Plaza, Upper Hill while the maximum allowable bearing capacity of 11673 kN/m² was encountered at a depth of 8 m at NCPD site.

Decomposed Nairobi Trachyte forms reddish brown plastic soil with very little gravel content. Tests show that the soil formed from weathering of Nairobi Trachyte settles when loaded and collapses when flooded; it can settle by 84 mm when flooded while loaded with 200 kN/m². An assumed bearing pressure of 150 kN/m² has been used on soils formed from Nairobi Trachyte. Some of the structures that have undergone distress in Langata and Karen areas are supported on these soils.

Fresh Nairobi phonolites are the most competent foundation materials in the study area. When fractured they are laterised along cracks and on further weathering they form a boulder bed. Nairobi Phonolite is encountered at various depths starting from 14 m. Tested unconfined compressive strength values for Nairobi Phonolite are generally moderate to high as observed at Central Bank of Kenya site. The UCS values range from 16 540 – 10 8000 kN/m² the while allowable bearing capacity is in the range of 4 467 – 29 463 kN/m². Because of the depth at which the phonolite occurs, only one foundation excavation (Hazina Towers) has exposed the upper portion of it. The lift shaft of the building is supported on the phonolite. The Nairobi Phonolite is suitable in supporting multi-storeyed buildings but care must be taken when excavating through it because the strong vibrations during drilling undermine the upper weaker materials. A borehole drilled in 1949 at "Mansion House" (BH No. C-792) backfilled by 20 m on drilling through the underlying phonolite. Later boreholes have used casing or bentonite slurry to prevent cave-in.

4.2.3 Shoring and underpinning operations done in Nairobi City

In this subsection, remedial and precautionary underpinning operations carried out in the Nairobi City are discussed. The cases discussed here are in no way exhaustive; they only comprise those obtained either from the Materials Testing and Research Department or from engineers and consultants that took time to present them during the research.

4.2.3.1 Remedial Shoring/ Underpinning

i) Residential house at Lavington Estate

A single storey residential house (S-2) in Lavington Estate had severe cracks that were opening downwards. Trial pits were dug adjacent to the exterior walls. The investigation determined that the structure was supported on fill that may have been a dumpsite and the fill was settling under the weight of the structure. To rectify the problem, pit underpinning was done in phases. Mass concrete pads measuring 1 m by 1 m by 1 m were prepared below the 1.2m deep foundation. The interior walls were also underpinned by drilling through the ground floor slab to access the foundation soils. After foundation underpinning, minor patching and repainting was done on the walls.

ii) Residential house at Garden Estate

The structure at Garden Estate (S-3) is a double storey residential house that had undergone distress; numerous cracks developed on the floor and partition walls. Trial pits excavated next to the foundations indicated that the subsoil at the site consists of 2.4 m layer of black cotton clay that is underlain by a laterite layer. The structure was supported on 1.5 m deep strip footings resting on 0.9 m layer of the expansive black cotton clay. During construction, the 1.5 m of black cotton soil was excavated in bulk and replaced with hardcore and concrete. The contactors probably thought that the expansive soil beyond 1.5 m would not be detrimental to the structure. Cracking started slowly and the size of the cracks increased with time. The floor was extensively damaged. To rectify the problem, excavations below the existing footings were made in phases and shored to provide safe working conditions. New footing bases were set at 2.4 m just on top of the laterite after which the foundation wall was constructed. The original footings were demolished and their loads were transferred to the new foundations. Ground floor slab and partition walls were all replaced. The upper floor was retrofitted and repainted.

iii) Residential house at Marurui Estate

The single storey residential house at Marurui Estate behind Safari Park Hotel (S-4) is built at a site with 6 m deep soft soils underlain by weathered rock. The contractor decided to use pad foundations taken to rock in order to minimize settlement. Design details indicate that columns were designed for every corner of the L-shaped structure and a ground beam was provided to carry the structural loads. On the long side of the structure, the columns were separated by 6 m span whereas on the other sides of the building, the column separation was between 3-4 m. The beam that was 6 m long started sagging and cracks developed. Afterwards windows and doors could not open.

During failure investigation, top marking across the cracks was done to investigate whether the cracks were opening. After periodic measurements it was observed that the cracks were not increasing in size. Trial pits were excavated to expose the substructure up to foundation level. The investigation revealed that the structure was supported on decomposed rock of variable strength. The structural design details indicated that all the columns were designed for 200 kN load. Due to the self weight of the ground beam and the loads from first floor slab, the beam on the distressed side was loaded with 40 kN-m moment that made it to sag causing cracks on the ground floor. To control the sagging and repair the house, a new column was introduced in between the existing columns to reduce the actual moment. The ground beam was shored to allow for excavation for the column up to firm ground. The sides of the excavation were strutted as the base was prepared. The column reinforcement was attached to the beam, bent and welded. The ground floor was demolished and set afresh; cracks on the walls were patched and repainted. The structure was observed for six months and no further problem was noted.

iv) Residential house at Karen Estate

The Karen project involved failure investigation for a one-storey residential house (S-5) that was extensively cracked on one side. Trial pit investigation showed that the columns were supported on pad foundations taken to firm soil while the structural loads were supported on ground beams. It was also noted that three columns on the cracked side were founded on laterite of lower bearing capacity and had therefore undergone differential settlement. Underpinning of the structure was done by excavation and placement of strip footings below the structure to provide the necessary rigidity. The load on the beams was distributed to the strip footings to relieve the columns. The method worked well and the retrofitting that was done was sufficient to restore the aesthetic value of the house. Similar conditions were encountered along Miotoni Lane (S-6) within the same locality and the structure was recommended for underpinning before the cracks could be patched and the house repainted.

v) Residential house at South C Estate

A masonry building belonging to Goal Kenya (S-8) was built to accommodate destitute children. Part of the house was built on non-structural fill in order to obtain the footprint required as it encroached on a hillside area bordering a river. For three years, the fill continued to consolidate moving downwards and outward together with the strip footing and interior slab. This movement initiated cracking in the floor, walls and beams. In order to determine the most cost effective solution, the engineers initiated a failure investigation to pinpoint the cause of the settlement. The study determined that one metre thick layer of hardcore had been placed on uncompacted fill to raise the ground floor level to avoid flooding but the fill was not retained on the lower elevation side. Due to the weight of the structure and thick hardcore, the poorly-placed fill moved outward and initiated outward movement of the hardcore material. Due to loss of support, the house was bulging out and, inevitably, distress occurred. The method of repair had to stop settlement of the footing as well meet limited project costs.

After evaluating several systems including concrete piles and ground modification, it was concluded that unconventional raft foundation would be the most efficient system. The solution involved excavation up to the base of hardcore and introduction of columns supported on pad footings at intervals of 3 m to retain the hardcore. The hardcore was then forced inside and beams were provided to tie the columns together after which a nice finish was given. Though the loose fill beneath the foundation system was not improved, the structure was considered to be robust enough to overcome differential settlement problems.

4.2.3.2 Precautionary shoring/underpinning

i) Loita House, Loita Street

Before construction of the 24 storey Loita House (S1), geotechnical investigation by means of boreholes was done to establish the depth to firm ground. The subsoil at the site consists of the following strata starting from the ground surface and working downwards: 3.2 m made ground layer; 3 m greyish fine sand layer; 1.6 m of weathered whitish grey tuff layer; 6.1 m dark grey porous tuff layer; 9 m of weathered brownish tuff layer; 7.7 m of decomposed grey tuff layer followed by phonolite up to the end of the borehole at 75 m. Groundwater was encountered beneath the made ground and the fine sand at the site was considered liable to movement. It was also noted that the made ground may have been placed after excavation of plastic clay during construction of the demolished building. From this information, it was apparent that the design of the structure had to include a method of underpinning of the adjacent structures to mitigate distress due to ground movement.

Foundations for the Loita House were designed as a raft taken to 12 m below street level so as to accommodate four basements. To underpin the adjacent structures, horizontal holes were drilled beneath the base of the foundations at intervals of 1 m; reinforcing steel was inserted and the holes were jet grouted using low mobility grout to avoid upward swelling of the ground especially at the base of the foundations. The process was repeated as the as the excavation progressed so as to control ground movement. The grouted material was considered to have formed a reinforcement mesh. The adjacent road was shored and the loose strata improved to allow ease of transportation and haulage from the site. This process enhanced the performance of a-12 m deep excavation ultimately supported by a concrete diaphragm wall. Waterproofing was done after a skin wall of 100 mm thickness was constructed. No distress was reported in the adjacent structures during construction or afterwards. The underground river reported from nearby sites was not encountered though enormous amounts of groundwater had to be pumped out during excavation.

ii) Nikisuhi Building, Industrial Area

Nikisuhi building (S-7) in Industrial Area that was under construction during the period of study is a commercial building with four storeys above street level and one basement. It is bordered on two sides by busy streets and two sides by existing buildings. Large amounts of groundwater were encountered in trial pits during site investigation. The structure was proposed to cover the entire site so exterior walls were designed as propped cantilever retaining walls. The permanent retaining walls were economically constructed using concrete diaphragm walls. Diaphragm walls combine into a single foundation unit the functions of temporary shoring, permanent basement walls, hydraulic (groundwater) cut-off, and vertical support elements. Because of this combination, they were considered to be an economical alternative in these circumstances. The pad foundation bases were cut 600 mm into the rock and the basement slab was placed on rock at 2.7 m depth. To avoid disturbance of the natural water flow resulting from obstruction by the basement wall, French drains were made below the foundation, 8'' perforated pipes were placed and connected to a sump from where water was pumped out. Because of the lateral pressures expected from the wet backfill, waterproofing of the basement structure using a chemical Cisa instead of membrane was carried out.

iii) Asili Cooperative House, Moi Avenue

The site at S-9 consisted of a two storey building with parking space behind. The new project involved two additional floors to the existing structure and conversion of the parking space into a 10-storey office building. Because of the conversion of parking space to office space, there was need to have a two basement parking probably taken to a depth of 7.0 m below street level. Given information from sites within the vicinity, the three major problems were anticipated: instability of the excavation, exposure of existing footings and groundwater flow. Detailed geotechnical investigation was undertaken to determine the ground conditions and in particular the depth and condition of footings. The ground investigation comprised a series of open-hole and rotary cored holes, together with a program of geotechnical testing. The ground conditions at the site were found to consist of up to 1 m of black cotton soil followed by weathered rock with a lot of groundwater flow. The pad footing foundation of the existing structure rested at 1.5 m depth below street level and was designed for two additional floors in the original plan.

When the position of the new structure had been determined, access requirements and optimisation studies on the comparative quantities and costs of earthworks, foundations and retaining walls indicated that a significant amount of bulk earth removal was necessary. Because of the inevitability of the weathered rock and groundwater in and around the area, it was concluded the process would undermine the adjacent building. The new structure was designed for strip footings and a support system consisting of piles cantilevered to the base of the footings was presented as an alternative to direct underpinning of adjacent building. The steel piles were machine driven to a depth of 10 m, 1 m away from the footings at every column position. Pile caps were provided on the piles. To transfer the loads from the existing foundations to the piles, horizontal cantilever beams were anchored on the piles and extended to the base of the footings. This method of underpinning was found to be sufficient and no distress has been observed several years after completion of the structure. A tank was provided to tap the groundwater for use within the building.

iv) I & M Building Kenyatta Avenue and Lonrho House Kaunda Street

The I & M Building (S10) is a 21-storey structure including four basements with the foundations set at 16 m below street level. The structure also comprises of a podium without a basement that borders Griflin Building. Examination of the structural plan for the Griflin House indicated that it was supported on pad footings resting at 1.5 m below street level. Excavation of the deep vertical cut for construction of I&M building could undermine the footings. Foundations for the tower block were designed as pad footings for supporting columns and strip footing for supporting walls. A fast-paced method of construction was adopted which permitted the earliest possible start on the tower structure so that subsequently work on the substructure and superstructure would proceed independently.

To construct the podium, two metre allowance was left from Griflin House to avoid exposure of the footings at the excavation face. Foundation bases adjacent to Griflin House were excavated in phases at alternate positions such that the existing footings were not undermined at two consecutive column positions. This was to ensure that the columns once established would provide support for the existing footings so that no ground movement would occur. Foundations for the podium were designed as pads anchored on piles to prevent uplift from the groundwater. After constructing the pad foundations, a retaining wall was constructed to act as permanent shoring for the old foundation. No black cotton soil was encountered at the site because the demolished structure may have built after soil was excavated. Groundwater encountered during construction was pumped out.

Lonrho House (S-11) is bordered by streets on three sides and a building on one side. The old structure comprised of a three storey building with no basement. To successfully complete the new building designed for eighteen storeys with a podium, three geotechnical boreholes were sunk outside the print area. The log of boreholes indicated that the yellow tuff supporting the existing structure extends to 5 m depth. This is followed by a 2 m thick layer of vesicular tuff with diagonal cracks beyond which is a layer of very weathered tuff. It was recommended that the new structure be supported on the pumiceous tuff that would act as a pressure distribution layer and thus reduce settlement.

An underpinning procedure was devised whereby before excavation, the walls were tied by beams. Excavation was done around existing footings; the footings were enlarged and the columns widened so that they would carry more loads. After widening the footings and columns, the additional 15 storeys were constructed without causing any distress on existing members. Problems experienced included: lack access of large geotechnical investigation and construction equipment to the interior of the house; installation of interior excavation supports; unique space constraints outside the site to support the equipment required for the facility; the shops were still in operation so vibration and noise were intolerable and delivery of construction materials could not be done during the day without crippling the city.

v) Hazina Towers, Monrovia Street

The Hazina Towers (HT) site (S-12) is bordered by two streets and two 6.1 m wide lanes. The vertical sided excavation for construction of the 21-storey structure was 100 m long, 41 m wide and 15 m deep. In June 1997, the development team of the HT commissioned a detailed site investigation for the foundations to the building. By then the excavation depth was at 4 m on approximately a third of the site. Six boreholes were drilled at representative locations and the report was published in October 1997. In addition to recommending pile foundations for the new building, the report stated that the vertical cuts were stable if supported by shoring props. By the time of the report, the excavation had reached full depth and had disrupted underground utilities leading to and from the Rattansi Educational Trust (RET) building. In December 1997, lateral and vertical cracks started to appear both inside and the outside of the RET building, doors became difficult to open and some steel windows jammed. At this time, the excavation had been completed and remained open for approximately two months.

After discovery of cracks at RET, shoring props were installed by the Hazina Towers Development Team (HTDT) to support the vertical face of the excavation at the back of the RET building. Eight steel struts were installed at intervals of 6 m to temporarily support the excavation across the entire length of the excavation behind RET. After 7th August 1998, the Site Office staff noted that cracks were forming towards the base of the face of the excavation supported by the eight props. On 19th August 1998 the vertical face behind RET collapsed taking with it four of the eight props. A section of the access lane also collapsed. Tenants using the stores and offices accommodated in the single storey structure below the first floor parking area at RET reported significant cracks in the walls and flooring, and the roof of these premises started to leak. An investigation indicated that a slip circle had developed and extended below the RET causing movement towards the excavation. The tenants were advised to vacate the wing.

Detailed physical inspection of RET building and interviews with the occupants of the building was commenced. The investigations revealed that RET was properly constructed under the control of reputable professionals and fully maintained and that it exhibited no structural damage before the start of the construction activities for the HT. History of the RET building indicates phased construction and consists of three wings. Koinange wing was designed for 9 floors. The first stage of construction of single storey building was in 1939; second storey in 1967; third storey 1970; fourth storey 1976; and, fifth storey in 1982. The two additional five-storey North and South Wings were constructed in 1989 with independent foundations. A reinforced concrete water tank was constructed on the ground level in 1993.

The tenants reported that the cracks first appeared as hairline cracks in the walls and the floors. They increased in number, length, and width as time passed. Eventually the cracks could be observed extending right through the walls from inside the building to the outside, cutting through the floor slab and tiles, along beams and floor slab joints. The cracks were wider at the top than the base indicating that the foundation had failed by hogging. Subsidence was caused by instability of the excavation for the basements and the foundations to the HT causing considerable damage and loss of revenue to the old building. To ensure that no further slippage of the sides of the foundation excavations or movement below the RET foundation, rock bolts were installed into the side of the excavation. This was largely successful in preventing settlement of the ground beneath. Further lateral restraint of the foundation materials beneath RET were provided by the construction of foundations and basements to the HT.

4.3 Final remark on the results of the study

After giving comprehensive results on the geological and geotechnical conditions of the Nairobi City subsurface, the research work intends to present a discussion based on each of the specific objectives of the study in the next chapter.

CHAPTER FIVE

DISCUSSION OF RESULTS

5.1 Spatial and temporal variation in groundwater rest levels

Shallow groundwater exists in the study area at depths ranging between 0.5-25 m. The groundwater is found to flow towards nearby rivers along the interface between overburden and weathered rock. Groundwater is also encountered in aquifers within all the geologic formations and at their contact surfaces from where it is extracted for domestic and industrial use. Boreholes in the same locality in Nairobi City can have different shallow water rest levels. Examples of shallow rest level variations are observed in geotechnical boreholes at the Kenya Pipeline Company in Industrial Area and National Social Security Fund (NSSF) in Karen area presented in Table 5.1 and Table 5.2 respectively. This is an indication of the occurrence of perched aquifers within the study area.

Shallow groundwater affects civil engineering structures in Nairobi City. For example, it was observed that entrapment of groundwater due to construction of deep basement at Hazina Towers could have a continuing effect in the stability of adjacent foundations. This was due to increased moisture content beneath the buildings caused by the interference with the natural groundwater flow by underground construction. Monitoring of the building was recommended to ensure that the foundations remain stable in the future and that the groundwater dissipates sufficiently to allow the soils beneath the foundations to regain their original strength. The consequences of lateral movement due to excavation for Hazina Towers were extensive property damage, construction delays, and expensive litigation.

Deep groundwater occurs in three main types of aquifers beneath Nairobi: aquifers in fractured volcanic rocks, aquifers in fluviatile sediments within volcanic rocks and aquifers in lacustrine sediments. In fractured volcanic rocks, aquifers with yields between 6-12 m^3/hr are common (Mailu, 1987). The tuffs and lavas give no supply being as a whole intact and impervious. The former may to some extent be porous but do not transmit water (Gevaerts, 1964). Important springs feeding the main rivers often issue from the contact between the soft and weathered rocks underlying impervious lavas (Saggerson, 1991). The aquifers of low yields are common in trachytes and phonolites where boreholes do not encounter any sediment as observed in north and south east of Nairobi (Mailu, 1987).

Table 5.1: Water rest levels at Kenya Pipeline Company in Industrial Area

Date measured	Borehole number	Depth (m)	Water Rest Level
11.08.99	1	15.9	12.65
11.08.99	2	10.60	2.90
11.08.99	3	10.80	9.70
11.08.99	4	10.35	5.10
11.08.99	5	15.70	4.50
11.08.99	6	11.70	6.05

Table 5.2: Water rest levels at NSSF plot in Karen.

Date drilled	Borehole ID	Depth (m)	Water Rest Level below ground level (m)
21/4-28/4/97	1	25.75	23.11
29/4-14/5/97	2	50.08	36.11
15/5-20/5/97	3	25.77	19.05
12/6-16/6/97	4	25.00	21.46
20/6-23/6/97	5	25.32	16.14
21/5-26/5/97	6	25.40	20.09
7/6-11/6/97	7	25.00	24.00
27/6-5/7/97	8	50.67	10.4
28/5-30/5/97	9	25.17	Dry
4/6-6/6/97	10	25.26	Dry

Borehole yield is the amount of water that can be drawn from a borehole per given period of time; in Nairobi City it is given in cubic metres per hour. The borehole yields are based on a 24 hour constant rate discharge pumping test performed immediately after a borehole is completed. In some boreholes, the yield is reported as the average volume of water that was pumped out per hour in a period of 24 hours. The amount of water recommended for abstraction is taken as one half of the average yield during testing (or smaller for boreholes in Karen and Langata). It is assumed that the recommended rate can be continuously pumped for a period of 12 hours without the aquifers being subject to depletion. Hove (1973) noted that the yield reported in old boreholes suffered the major limitation of representing the pump capacity. Also, the yield expressed is a function of the capacity of individual aquifers.

The occurrence of groundwater in Karen-Langata area is from Kerichwa Valley Series sediments and tuffs, at the contact zone of phonolites and trachytes and within the Athi Series. From available records, more than one aquifer has been struck in the horizons between Nairobi and Mbagathi trachytes and within Kerichwa Valley Series. No appreciable increase in yield is obtained in deeper boreholes in this region as is observed in boreholes within 1 km radius presented in Table 5.3. However in other localities, yields increase with depth as illustrated by boreholes within a radius of 2 km in Industrial Area presented in Table 5.4. The first aquifers encountered in Karen-Langata area are semi-confined with piezometric rises of between 3 to 9 m above the first water strike (Table 5.3). Piezometric rise is the difference between the water struck level and the water rest level.

Table 5.3: Low to moderate yields in boreholes in Karen

B/H No.	Depth (m) asl	Water struck level (m) asl	Water rest level (m) asl	Tested yield (m^3/h)
1715	1661.9	1720.0	1780.8	0.68
1026	1795.0	1811.3	1814.3	5.5
1023	1795.0	1814.0	1821.0	2.7
1038	1804.0	1821.0	1825.0	2.73
1415	1696.5	1789.9	1781.0	3.72

Table 5.4: Increase in borehole yields with depth in Industrial Area

B/H No.	Depth (m) asl	Water struck level (m) asl	Water rest level (m) asl	Tested yield (m^3/h)
11317	1472	1554,1528	1550	3.27
10122	1470	1574,1554,1475	1552.9	5.7
10662	1445	1612,1478	1531.7	16.4
14856	1440	1577,1485	1531.6	12
13124	1423	1531,1477	1532.3	16
13087	1407	1470,1428	1523.7	9
13086	1391	1475,1433	1536	15.9
13220	1368	1456,1424	1505	18

According to van Tonder *et al.* (1998), there are two important rules that need to be kept in mind when determining the sustainable yield i.e.: the total abstraction from a borehole should be less than the natural groundwater recharge, and secondly, a borehole should be pumped in such a manner that the water level never reaches the position of the main water strike (normally associated with a fracture). Should this happen, the yield will inevitably be affected and the borehole would eventually dry up.

Specific capacity is a valuable tool for assessing the available groundwater resources. It is the ratio of yield to drawdown and is expressed in m^3/hr/m of drawdown (Hove, 1973). Some boreholes in Industrial Area have specific capacities in the range of 4.8 -8.2 m^3/hr/m while low yielding boreholes in Karen have specific capacities less than 0.1 m^3/hr/m. Hove (1973) obtained specific capacities in the range of 0-1.25 m^3/hr/m in volcanic formations in central Kenya, with local pockets in which the capacity ranges from 1.25-50 m^3/hr/m.

Hove (1973) states that thick and laterally extensive saturated aquifers suffer very limited drawdown after releasing large volumes of water, thus showing high specific capacities, whereas unsaturated or thin and laterally restricted aquifers (e.g. perched aquifers) suffer considerable drawdown even when small volumes of water are abstracted. Generally, therefore, higher specific capacity indicates greater the availability of the groundwater. In this regard, there will be more water available in east Nairobi in the long term as compared to Karen-Langata area. If circumstances permit, more settlement will be expected in the Karen-Langata area as a result of depletion of aquifers.

Boreholes along Ngong Road in Kilimani and Woodley Estates have rest levels below 100 m because the aquifers are generally deep. Any aquifers above 100 m depth are perched. Table 5.5 presents some hydrogeological information on boreholes drilled in different decades in Woodley and Kilimani areas.

There is a general tendency of boreholes in a locality to belong to the same decade probably due to severe water scarcity in that locality at that particular time. For this reason, rest level records for some localities are missing during certain decades. Examples are majority of boreholes in the City Centre that were drilled in the 1990-1999 decade and Karen and Ruaraka areas dominating boreholes in the 1940-1949 decade (Figure 4.6). A summary of the water rest level changes depicted on the Surfer contours and the averages calculated from records for the various localities is presented in Table 5.6.

Table 5.5: Hydrogeological information on boreholes in Woodley and Kilimani

BH ID.	Owner	Depth (m) asl	Water struck level (m) asl	Water rest level (m) asl	Tested yield (m^3/h)
C-1750	Moi Girls School	1420	1706,1570	1570	6.8
C-2987	Impala Club	1415		1603	4.5
C-3775	Royal Golf Club	1399	1534,1409	1604	11.4
C-11592	Uchumi Hyper	1470	1668, 1528, 1472	1577	4.6
C-13916	Blackrose Apartment	1440	1470	1495	12
C-12902	Yaya Towers	1420	1499, 1456	1554	12

Table 5.6: Water rest levels (asl) in different decades and the calculated average drop

Locality	**1927-1939**	**1940-1949**	**1950-1959**	**1960-1979**	**1980-1989**	**1990-1999**	**2000-2009**	**Average drop (m)**
Karen	1788	1793	1803	1782	1828	1805	1781	7
Langata	1733	1738	1709	1716	1735	1712	1698	35
City Centre	1644	1641	1605	1609	1556	1562	1542	102
Lower Kabete	1752	1743	1710	1750	1702	1695	1676	76
Industrial Area	1600	1595	1551	1621	1604	1534	1515	85
Ruaraka	1553	1524	1507	1527	1546	1533	1484	69
Average drop from records (m)	**5.59**	**5.51**	**6.81**	**-10.57**	**7.77**	**24.2**	**29.46**	**79.34**

From Table 5.6, it can be observed that deepest rest levels have been falling except 2000-2009 decade when boreholes in east of the City Centre encountered extensive Athi Series aquifers with large storage capacities. The largest fall in rest levels has occurred at the City Centre in which old boreholes had rest levels at 1644 m above sea level while new boreholes have water rest levels at 1542 m above sea level. Although boreholes in Karen area have low yields, they seem to have adequate replenishment through the many faults that affect the area and thus show rise and fall in rest levels throughout the investigated period.

Whereas the Surfer contours show no rest level drop in 2000-2009 decade for boreholes in east Nairobi, the averages from records indicate a drop of 29.46m. The deepest rest level contour on Surfer by 1939 was 1500 m while by 1999 rest levels of 1380 m were recorded. The average drop calculated from the last column in Table 5.6 is 62 m for the six main abstraction areas. The cumulative average drop calculated over the 80 years of abstraction across the entire study area is 79.34 m, of which 53.66 m (67.6%) has occurred in the last two decades. GIS indicates an average rest level drop of 62 m between 1940 and 2010.

5.2 Subsurface profile and probable settlement due to groundwater extraction.

Subsurface profiles from across the study area indicate that laterites or fractured rock underlie overburden soils. Most structures are taken to rest on the fractured rock or laterites and during excavation water is reported to flow at high velocity at the interface between the overburden soil and the laterites/fractured rock sometimes flooding foundation excavations. A few sites report elevated sulphate and chloride levels in the water. The origin of the laterites is the yellow tuffs that are found intact beneath the black cotton clay at some sites. Depending on the site, clay with a total thickness ranging between 5 m and 8 m will be encountered if the excavations are done to 30 m depth. The clay is sandwiched between fresh and weathered tuffs that serve to spread the loads from structures. Between 30 m and 80 m depths are clay layers of very small thickness relative to the thickness of the flesh lavas.

Several volcanic flows can be observed within each geologic unit, some with weathered or vesicular upper contacts. In between the flows are old land surfaces of different thicknesses. The most prominent and consistent old land surfaces occur within Nairobi Phonolite and at the contact between Nairobi Phonolite and Athi Series. The old land surfaces are made of laterites, sands, clays, silts and conglomerates and have a thickness range of 8 m to 36 m. Mbagathi Phonolitic trachyte is heavily fractured and displaced at many locations and tongues into the Athi Series (Saggerson, 1991).

The calculated maximum settlement that might result from groundwater exploitation is 5.9 m at Ngong-Kiserian area while the minimum settlement is 0.034 m at Kibera/Woodley area. The average settlement due to elastic and inelastic compression is 2.1 m. The probable settlement is higher at Ngong'- Kiserian area because of the presence of large unconsolidated deposits associated with faulting. The magnitudes of settlement calculated indicate that although most of the world settlement that is related to groundwater drawdown is common in sedimentary environments, probable magnitudes of 5.9 m should serve as a wakeup call to put up measures that can mitigate probable subsidence and the related consequences on civil engineering structures.

5.3 Reasons for sudden changes in subsoil type

Some on-site boreholes such as those at Mbagathi and Ololua ridges in Karen indicate the effects of the fracture traces in their borehole yields. Groundwater flows in the direction of the local gradient; in fractured rock, the gradient is defined by the orientation of joints, faults, bedding planes and shear zones. Wells that intercept one or more fracture zones typically exhibit a higher yield (Batte *et al.*, 2008). Mbagathi Ridge in Karen was therefore confirmed to be a fault as boreholes on either side of it have different profiles and yields. The presence of pollutants in a deep borehole at Hillcrest High school in Karen was also taken as an indication of the presence of faulting and reason for variable subsurface profiles.

The existence of very weak materials at the Lands Office site at Community and the fractures traced through Upper Hill Estate and City Centre seem to suggest the presence of buried grabens in many parts of the study area. On the geological map, most fracture traces are covered by deep soft soils and therefore have no surface expression. It is therefore correct to say that some swamps were formed in depressions related to faulting. Analysis of the depths at which the fracture traces are observed in boreholes indicates that they become deeper eastwards. Hove (1973) noted that the complex pattern of faulting and cross faulting in the Rift Valley can be resolved into a simple system trending NW-SE in the southern parts of the area and later swinging right until they are aligned almost N-S. This trend of faulting is maintained in the fractures traced in the study area that is outside the rift valley.

Geophysical surveys can be used to reveal these buried fractures. Vertical electrical sounding in resistivity survey involves collection of subsurface information at log steps. Using the Schlumberger array, the first electrode distance from the centre is 1.6 m the next step is 2.0 m then 2.5 m, 3.2 m and so forth. The incremental steps at depth are large, for example from 250 m the next step is 320 m. For this reason, the lower part of the profiles undergoes a more restrained modelling due to lack of data. Also, with depth, measured potential values are influenced from a relatively wide area of the subsurface as the current spreads out laterally. This means that the resulting resistivity values describe an average from a relatively larger volume (Solberg and Sandven, 2008).

When the resistivity data are modelled, the subsurface is separated into layers whose resolution is determined by the electrode incremental step. This means that the resolution in the profile is better if the electrode spacing is small and in the upper parts of the profiles. Because of decrease in resolution with depth, the Earth Imager layer models presented in this study display information corresponding to 0.55 of the measured depth. For example, if the electrode distance was 250 m, as was the case in Figure 4.23, then the resistivity model will display layer information up to 176 m. With depth, the Earth Imager modelling fails to distinguish the layers resulting in thicker layers and beyond 0.55 of the electrode distance it cannot detect any layers. To obtain valuable information for a certain depth, it is therefore important to measure deeper.

From the field work, it was observed that the measured resistivity values in Nairobi are generally less than 200 ohm-m. This is because they represent the average along the path travelled by the current that includes thick weathered or decomposed layers sandwiched between intact layers. Higher resistivity values are recorded for near surface layers and are probably so because the field work was carried out at the end of a long dry season. In general, resistivity values for sites with dominance of clays and decomposed materials are very low (<30 ohm-m). According to Telford *et al.* (1990), resistivity values for bedrock can be several thousand ohm-m. Ramamurthy (2008) found that resistivity values of less than 75 ohm-m in soils and less than 200 ohm-m for hard/fractured rock indicate the presence of water in the voids.

From the variety of curves obtained in the resistivity survey, it can be concluded that the subsurface profile throughout the depths investigated is variable. The most common subsurface profile is HAKQ type that represents deep soft formations. In all, the transition from weathered to fresh rock resistivity values appears gradual and it is sometimes difficult to separate weathered material from fresh rock. The layered earth model is actually very much a simplification of the many different layers which may be present. The various equivalent solutions that can be generated by a computer programme should therefore be carefully analyzed. When interpreting the VES with computer software it is important to realize the following two effects:

1. Equivalence: equivalence is the problem of having different interpreted computer models for the same resistivity curve. This is the result of the fact that usually more than one solution is possible e.g. a relatively thin layer with a high resistivity may give the same result as a thick layer with a slightly lower resistivity.
2. Suppression: when the thickness of a layer intercalated within a sequence is relatively small it may not be noticed in the resistivity curve and is 'suppressed' and therefore not sensitive to the computer interpretation. Nevertheless, where justified (e.g. when it is known to exist from borehole data) this 'invisible' layer may be introduced in the interpreted model (Batte *et al.*, 2008).

5.4 Engineering properties and bearing capacities of the subsoil

Measurements of mechanical rock and soil properties generally require well prepared samples and testing is time consuming and expensive. For this reason, majority of the geotechnical investigation tests in the study area have been carried out on materials deemed suitable for supporting the foundations or those that look suspicious or have performed unsatisfactorily in some past projects. The gaps in knowledge about the nature of the rest of the subsurface materials and their expected performance have been bridged by analysis of logs recovered from water supply wells and from construction excavations. Determination of mechanical properties of materials that have not been geotechnically investigated has been done by correlations between mechanical properties of subsoil encountered in boreholes and resistivity measurements.

Most of the geological formations underlying Nairobi exist in decomposed, weathered and fresh forms making foundation design on them challenging. According to Look (2007), there is approximately ten fold increase in allowable bearing pressure from an extremely weathered to fresh rock. Sowers (1979) states that when foundations of multi-storeyed buildings are constructed on rock mass that is inhomogeneous and discontinuous, a greater degree of conservatism should be exercised in the estimation of load carrying capacity and the resulting deformation of rock mass.

Sowers outlines the characteristics of the subsoil that lead to the use of a higher factor of safety: heterogeneous nature; existence of highly fractured and decomposed rocks that are subject to development of rupture surface when loaded; presence of very weak layers underlying strong layers, and; the presence of groundwater. Because of the heterogeneity of the Nairobi subsurface, a factor of 10 was applied on the ultimate bearing capacity to obtain the allowable bearing capacity. The bearing capacities and allowable pressures on the various geological units tested are presented in Appendix 10.

Foundations constructed at greater depths may increase the ultimate bearing capacity of the foundation. The improved capacity is due to a greater passive resisting force and a general increase in rock mass strength with depth. The increased lithostatic pressure closes discontinuities, and the rock mass is less susceptible to surficial weathering. Occasionally, deeper burial may not be advantageous. A region with layers of differing rock types may contain weaker rock at depth. In these instances, deeper burial may even decrease the bearing capacity.

Records indicate the following design pressure ranges for low-rise and medium-rise structures on the various soil types: laterite 150-400 kN/m²; weathered rock 150 -250 kN/m²; red soil 150 -250 kN/m², soft tuff 200 -250 kN/m²; clay 150 -225 kN/m², and, stiff clay 180-200 kN/m². It is apparent that the values used differ from designer to designer and that designers are more confident on the behaviour of red soils under load than for soft tuff and weathered rock. Also noted is the fact that structures in Eastleigh area are designed to rest either on rock, laterite or soil and therefore there is a wide range of design values that are used (Appendix 9). It is also noteworthy that smaller design values are used in Dagoretti, Thome and Lavington because they are known to have deep soft soils.

Together with borehole logs, resistivity profiles seem to provide a good indication of the material properties of the subsurface. Results of this study indicate that the profiles and borehole logs relate very well and can thus be used to bridge the gap in knowledge about the subsurface materials. Comparison of the layered resistivity models and borehole logs shows that when the subsurface materials are weathered, regardless of the origin, their resistivities will be very close and as such will appear as the same geoelectric layer on the resistivity curve. Resistivity curves for sites with similar subsurface materials in Nairobi City have similar shape and hence can be used to predict similarity of the subsurface profiles.

Ramamurthy (2008) also found that geophysical interpretation of mechanical properties of the subsoil shows significant changes and trends in the subsurface that match very closely with well log data and resistivity log. Limitations of the geophysical method are that the models used for interpretation of VES apply to homogeneous, semi-infinite layers, as they may occur in sedimentary areas. Using the same models in areas with sometimes extremely variable thickness and where the top of the layer can be very disturbed is an extrapolation, and results such as mathematically interpreted true resistivities and thickness may be subject to caution (Batte *et al.*, 2008).

Kahraman and Alber (2006) correlated electrical resistivity values obtained from electrical impedance spectroscopy measurements with the corresponding physico-mechanical rock properties for the eight different samples cored from a fault breccia. They found significant correlations between resistivity and physico-mechanical rock properties. Recently, Vipulanandan and Garas (2008) investigated the electrical resistivity and mechanical properties of carbon fibre-reinforced cement mortar. They developed some empirical relations for specific electrical resistivity to unit weight, Young's modulus and pulse velocity.

Kahraman and Yeken (2010) performed laboratory experiments of resistivity on igneous rocks saturated in brine and related the measured resistivity with tested values. They obtained values of UCS ranging from 50.2 MPa for volcanic bombs to 202.9 for basalts but concluded that the effect of inclusion of stronger rocks needs to be investigated. The UCS values calculated from resistivity measurements from Nairobi City range from 20 MPa to 54 MPa (Appendix 7). These ranges of values therefore are close to those obtained for the volcanic bombs. A formula given by Kahraman *et al.* (2006) produces 50 MPa UCS values when the resistivity is close to zero, implying that a compressive strength is obtained even for water.

Since the ultimate and allowable bearing capacities calculated by the Ramamurthy (2008) formulae are more than tenfold higher than the tested values, they cannot be used for estimating the bearing capacity of the ground for design purposes but can be used to study the overall homogeneity of the subsurface. They can therefore be confirmatory for areas investigated by boreholes or other direct means. When the values are compared with tested properties established from core logs, they can give an insight into the subsurface.

5.5 Strata liable to collapse at construction sites or settle as a result of physical weight of high-rise buildings

Most low-rise structures are supported on footing foundations taken to laterite or weathered rock at 1.2 m to 1.5 m depth. Old low-rise structures within the Nairobi City Centre were designed to carry at least two additional floors. When requests for increasing the height of the buildings are made, the City Council of Nairobi requests that the engineers confirm the state of the existing foundations and determine whether they are capable of supporting additional load. They also request for a specification on how the adjacent structures will be supported during construction. The important design criterion is the requirement to avoid encroachments on existing buildings in the close vicinity of the new heavy building.

The yellow tuffs sometimes extend up to depths of 6 m below the ground surface and therefore support most of the shallow foundations structures set at 1.5 m to 2 m depth. Excavations made through intact yellow tuffs are fairly stable and do not need any form of support. Darker yellow tuffs are described as brown tuffs at some sites and give rise to brown highly plastic clays when weathered. From test results, intact yellow tuff has allowable bearing capacity of 500-700 kN/m^2 and when horizontally and diagonally fractured with clay infills, the bearing capacities are in the range of 350-500 kN/m^2. Decomposed yellow tuff has allowable bearing capacities in the range of 250-300 kN/m^2. Design pressures on the yellow tuff have been in the range of 250-400 kN/m^2.

A number of classification tests have been carried out on soils formed from decomposition of yellow tuffs because they physically look suspicious. Completely decomposed yellow tuff forms yellowish-brown silty clay with high natural moisture content, high to extremely high plasticity and expansive properties. When dry, the clays are stiff and fissured. On grading, the soils are found to consist of large fractions of either silt or clay and small fractions of sand and gravel. Only one grading curve for this soil has a sigmoid shape. The fine fraction in the decomposed yellow tuff is washed away during rotary drilling and therefore some geotechnical borehole logs indicate no recovery, meaning that it was not recovered as a core. Coarser- grained yellow tuff is weathered to lateritic clay with physical characteristics of laterites but with low bulk densities and sensitivity to moisture changes. Structures supported on the lateritic clays have undergone distress of varying magnitudes depending on the amount of moisture accessed under load.

Yellow agglomeratic tuffs have been found to be soft and unstable when in contact with water and when weathered they form gravelly clay. Because of the poor bonding between the fine matrix and the boulders, excavations done through the agglomeratic tuff are susceptible to collapse when left open. The agglomeratic tuffs at the Hazina Towers were undermined while excavating through the Nairobi Phonolite leading to distress at the Rattansi Educational Trust building despite the fact that the excavation was more than 6 m away. The situation was aggravated because the excavation remained open for a long time during which there were incidences of heavy rainfall. A method of shoring of excavation through the agglomeratic tuff should be provided for to avoid development of slip circles.

The presence of stronger layers of Kerichwa Valley Series tuffs at the City Centre serves to reduce the total settlement in civil engineering structures. The designs are also done taking into consideration the presence of weathered materials. For instance at Lonrho House, diagonal cracks were observed on the yellow tuff formation that extends to 5-6 m and is underlain by thick weathered rock. Out of settlement consideration the building was recommended to be founded on the yellow tuff. This yellow tuff would serve to spread the loads to the underlying weathered layer. If the cracks appeared wider during foundation excavation, cement grout could be injected into the cracks. Similar load-spreading effects are expected when foundations are set on the black tuff beneath which is yellow tuff, sometimes decomposed to clay.

Real-time settlement measurements at KICC and Bruce House buildings indicate that settlements due to the weight of skyscrapers are small if the structures designed and constructed well. In practice, differential settlements between footings are generally controlled, not by considering differential settlement itself, but by controlling the total settlements predicted using estimate of elasticity. This approach is largely based on correlations between total settlements and differential settlements observed experimentally and lead to a limitation of 4 to 8 cm in total settlement under footing as stipulated by Canadian Foundation Engineering Manual (Fenton *et al.*, 2003). Because of the wide variety of soil types and the possible loading conditions, experimental data on differential settlement of footings founded on soil is limited. With the aid of modern high speed computers, it is now possible to probabilistically investigate differential settlements over a range of loading conditions and geometries (Richardson, 2005).

Spread footings of various shapes and dimensions are the most commonly used types of foundations in the city. Pad foundations combined with floor slab have been used where groundwater is a concern. For areas where the soft subsoil is deep, pad foundations are taken below the soft soil and the columns are tied at intermediate sections to avoid buckling. All the loads are supported on ground beam and the floors are suspended above the ground. Because most of the shallow foundations are set at depths of less than 2 m below the ground level, a method of supporting the foundation soil or the foundations themselves should be included in the design when making vertical cuts beyond the foundations.

Customary raft foundations are adopted on very soft ground or on sensitive soils. For small structures, the rafts are placed just after removing vegetable soil while for larger structures they accommodate basement. In most cases, hardcore is placed beneath the raft to avoid wetness of the ground and control swelling. The ground floor is raised above the ground because such areas are also prone to stagnant water due to low permeability of the soft soils. A few sites recommended for raft foundations utilized pad footings with conservative bearing pressures e.g. S/No. 4, 8, 16 and 30 in Appendix 8.

Point bearing piles have been used when the subsurface is soft and deep or when large inflows of water into the construction excavation are expected. Examples are those used to support some houses in Nairobi South B estate, Kijabe Street at the City Centre, Nairobi South, Karen and on Kabete Red Clay. Piles penetrating through the agglomeratic tuff were recommended for some sites though they were not used; these piles were considered as purely frictional. Piled rafts have been utilized on the podium sections of some buildings such as I &M building along Kenyatta Avenue. This was to overcome uplift pressures from groundwater flowing beneath the rafts. Settlements in structures in Nairobi City have been largely controlled by use of conservative design pressures. No intolerable settlement has occurred in skyscrapers because of use of appropriate foundation types and design pressures.

5.6 Causes of defects in structures

The cases of distress presented in this study were initiated by the owners of the affected structures. The causes of defects are not necessarily representative of the generality of building defects in the city; but they should be indicative of those for which there was no obvious explanation to the people directly involved and on which an independent opinion was sought. Most of these cases involved footing foundations up to 2 m depth. The remedial measures have ranged from minor retrofitting to extensive underpinning.

Bulging floors, cracked walls, and jammed doors and windows are all signs of foundation distress. The problems occur due to adjacent excavation or when only part of the foundation heaves or settles causing cracks and other damage. Non- uniform settlement (dishing) can result from either non -uniform stresses acting on homogeneous soil as in the case of the structure at Marurui Estate or uniform stresses acting on non-homogeneous soil conditions as in the case of the residential house at Karen Estate. The magnitude of differential settlements is affected greatly by the non homogeneity of the natural soil deposits and also by the ability of structures to bridge over soft spots in the foundation. From field observations, settlement cracks are nearly always vertical while soil bearing failure cracks occur at each side of a portion of the foundation wall that is undergoing downward movement (Richardson, 1991).

Plotting sites with distressed structures on the geological map indicates that almost all areas covered by the alluvium, clays and swamp soils have reports of distress in structures with the exception of Dagoretti area. The alluvium comprises mainly of slightly overconsolidated and compressible silts that cover former and current river valleys. Along Kirinyaga Road, the deposits are clayey silt and more liable to collapse as occurred during construction of Parsonic Hotel. Similar clayey silts are found underneath View Park Towers, Grand Regency Hotel and Continental House. Elsewhere in the world (Seed, 1970), the silty clays have been found to be liable to liquefaction during earthquakes. Old structures supported on these soils in Garden Estate, Karen, Madaraka and Nairobi South areas have experienced distress more than five years after construction. According to Richardson (2005) buildings that experience settlement several years after construction may have been overloaded.

In Karen, the alluvial soils are closely related with faults for instance at the Karen Police Station and Warai North Road where they are associated with distress in structures. The colour of these soils varies from yellow to white to brown to black and red. This shows that colour cannot be a means of recognizing the soils at a site. Before constructing in the vicinity of alluvium, clays and swamp soils, it is important to do a site investigation and test samples for Atterberg Limits, particle size distribution and for collapse and swell to avoid designing foundations that are likely to fail (Goldsworthy *et al.*, 2004).

Excessive lateral displacement and the associated ground settlement are often the primary cause of damage of nearby buildings (Kany, 1977). Buildings adjacent to the construction site may settle well beyond an acceptable limit if excavation is carried out without any protection measures being taken. To increase the passive resistance of the soil mass, soil improvement and diaphragm wall can be used as in the cases of Loita House and Nikisuhi Building. But to evaluate the effectiveness of these systems, it is necessary to understand the sources of the lateral pressures. Shoring is effective when loading is not excessive and excavations are not so deep. The shoring system should provided as the excavation proceeds and it is important not to leave the excavation open for long after it has reached full depth.

Deformations resulting from adjacent construction can be caused by: build-up of pore pressure as a result of obstruction of natural flow of water due to construction of basement; settlement due to lowering groundwater table required during basement excavation for a new building; heave due to stress relieve of a basement excavation or demolition operation; settlements around a new building; and, heave after the end of dewatering when the water table rises to its original position (Burland *et al.*, 1972). These different reasons make the prediction of movements of existing buildings a complex task which depends, besides others, on the construction sequence and its quality. Although practicing engineers well know that deep foundation construction frequently has detrimental effects on adjacent structures, very few case are described in literature. Lack of case histories is due in part to the possible legal action seeking if such cause and damage is acknowledged or the fear of losing tenants of the owner also acknowledges (Lambe and Whitman, 1979).

Seismology data prior to 2007 indicates very minimal earthquake activity in the general area of Nairobi but high activity in the Ongata Rongai/Ngong areas (Wanyumba, 2001). An earthquake of magnitude 4.4 occurred in Ongata Rongai/Kiserian area (20 km from the City Centre) on 26th March 1997 otherwise all other locations have experienced earthquakes of magnitudes below 4.4 on Richter scale. The most seismically active centre in Kenya is L. Magadi that is 100 km south of Nairobi City. In 2007, several earthquakes and tremors with the epicenter at L. Natron (south of L. Magadi) affected the city. The largest earthquake of all measured 5.3 on the Richter scale according to USA Geological Survey.

When the hypocentral distances were plotted against magnitude in Figure 3.4, they showed that Nairobi City Centre is unlikely to experience liquefaction. However, fault zones nearer to the epicenters can be affected thus experience defects in structures. The most susceptible areas are Ongata Rongai, Nkoroi and Mbagathi where faulting and cross-faulting is intense (Saggerson, 1991) So far there is no link between defects in structures and earthquakes since most of the reported structures are in the east and the south of the City Centre.

5.7 Final remark on discussion of results

After giving a discussion on each of the specific objectives, the research work will present the major conclusions, contributions to knowledge in this field as well as to future recommendations in the next last chapter.

CHAPTER SIX

CONCLUSIONS, CONTRIBUTION TO KNOWLEDGE AND RECOMMENDATIONS

6.1 Conclusions

Below is a summary of the conclusions made regarding each of the specific objectives.

6.1.1 Variation of water rest level in space and time

Shallow groundwater (0.5 – 5 m deep) exists in majority of the sites to the east of the City Centre. It is also encountered within depths of 6 m in Lavington, Kileleshwa, Kilimani, Thompson, Westlands, Bernhard and parts of Karen estate. Disruption of groundwater flow to nearby rivers during construction excavation causes softening of intended foundation bases and other construction hazards. The main methods of minimizing the flow problems have been sump pumping and placement of hardcore to allow the water to flow in its natural course to nearby rivers. There are groundwater flow problems into basements of some buildings within the City Centre such as City Hall Annexe among others and hence a possible source of distress resulting from lateral pressures.

Increase in the number of water supply boreholes in the study area has been in direct proportion to population growth. Records indicate that continuous rest level monitoring has been carried out in 6 continuous years in the 80 years of active abstraction. In the absence of continuous water level measurements, the initial rest levels measured after borehole drilling were used to study the spatial and temporal variations. The boreholes were classified according to the decades in which they were drilled.

The results of groundwater rest level analysis on Surfer and GIS as well as the calculated values indicate that the levels have been falling consistently. GIS shows average rest level drop of 73 m while Surfer shows deepest contour drop from 1500 m by 1939 m to 1380 m by 1999. The rest level drop calculated from records is 79.34 m for the 80 years, of which, 53.66 m (67%) has occurred in the last two decades (1990-2009).

From the surfer contours, 1960-1979 decade saw rise of rest level by about 40 m; the averages calculated from records indicate a rise of 10.57 m. The rise in average rest level during this period occurred due to controlled abstraction and restriction in drilling (Mwangi, 2005; Foster and Tuinhof, 2005). This implies that although most of the Nairobi aquifers are confined, recovery of rest levels can be attained by strict management of groundwater resources. Property developers should be encouraged to harvest rain water to reduce reliance on groundwater so as to ensure that drawdown does not occur and lead to surface subsidence.

6.1.2 The subsurface profile

Most of the water supply borehole logs at the City Centre indicate that the near surface subsoil is comprised of alternating weathered and fresh tuff layers up to 16-20 m. Between 20 and 30 m, layers of fresh Nairobi Phonolite or weathered agglomeratic tuffs are encountered. The deepest geotechnical borehole at Loita House indicates that very strong Nairobi Phonolite exists at 59 -75 m depth. Most formations at 78 -130 m depth are decomposed. Several other weathered layers are found up to 470 m where the metamorphic rocks are encountered at the City Centre. The thickest decomposed material is recorded at ICRAF Gigiri (103 -304 m) while at Rosslyn area, the subsurface consists of heterogeneous sediments, sand and silt without any intervening tuffs. In general, the total thickness of weathered materials in the city is larger than the total thickness of fresh and fractured rocks.

Geotechnical investigations indicate that the Nairobi subsurface is heterogeneous, consisting of various subsoil types that have been characterised by data. Delineation of these geologic materials and estimation of their engineering properties has been done from limited data points of direct site investigation. In this study, the engineering properties of the subsurface materials have been described for various localities within the city.

The qualitative and quantitative properties most relevant for geotechnical considerations are presented in tables and figures (Koloski *et al.*, 1989). The properties are based on compilation of unpublished information from field and laboratory tests performed over many years and a few original research tests. Because of the extremely variable nature of the geologic materials, the ranges presented in the tables should be considered representative, but not necessarily allinclusive. Where ranges are indicated, it is estimated that majority of the observations will fall within the same ranges.

Three to six aquifers with total thicknesses ranging between 28 m and 60 m have been encountered in water supply boreholes. The aquifer system comprises unconfined, semi-confined and confined aquifers in fluviatile and lacustrine sediments as well as fractured volcanic rocks. Aquitards (clays) that are subject to permanent compaction as a result of groundwater drawdown occur at several depths; at some localities they commence at the ground surface. In the confined system, they occur within old land surfaces and Athi Series at average depths of 78 m below the ground surface and have an average thickness of 53 m.

Settlement that can result from groundwater withdrawal was calculated for confined aquifers and clay aquitards. Groundwater rest levels were found to have dropped with an average of 73 m in the last 80 years and a probable estimated settlement of 0.034 m to 5.9 m could result from groundwater depletion from aquifers and clay aquitards over a long period of time. Between the ground surface and the clay aquitards are the dense Nairobi Phonolite and/or Nairobi Trachyte. Flexure of the thick Nairobi Phonolite and Nairobi Trachyte layers will have to occur for this settlement to affect the structures at the ground surface. So far, no defects in structures in Nairobi City can be related to groundwater drawdown. Settlement due to groundwater withdrawal has caused numerous problems in cities such as Mexico (Coduto, 2001), Shanghai (Xu *et al.*, 2004), Bangkok (AIT, 1981), Venice (Barnes, 2000) and Osaka (Mimura and Young, 2008) among others.

6.1.3 Reasons for variable subsoil

The results of this study show that Nairobi City was once a swamp that resulted from drying up of a large lake or a series of lakes. Some of the volcanic materials were formed from graben-forming collapse. These results lead to a conclusion of the two possible reasons for variable subsurface profiles: many centres of volcanic activity leading to formation of subsoil with different geotechnical characteristics and several grabens concealed by swamp soils and deep alluvium.

The resistivity measurements give a continuous and relatively detailed picture of the subsurface in such areas but are supplement not substitute for geotechnical investigations. In an area without previous geotechnical investigations, the resistivity results will constitute a basis for further exploration and suggest optimal locations for geotechnical drilling. Resistivity survey can also be used to confirm the presence of faults at a site if the electrode spread is done along and across the suspected faults.

6.1.4 Engineering character of the critical subsoil layers

The character of the subsurface materials in Nairobi City is related to their geologic origin and depositional environments. Deposits that are erupted into the air and away from the source are always fine-grained while those deposited in water exist in layers of varying texture. Due to the presence of large boulders in agglomeratic tuffs, earlier researchers concluded that the materials may have been erupted through nearby faults since such heavy materials cannot be deposited so far away from the vent. Existing literature actually shows that agglomerates are deposited near vents and are variable in character depending on the fraction of the fine matrix and bonding of boulders. A summary of the engineering properties of the subsoil encountered at building sites is gives below.

Character of alluvium and clay

This group of soils is mainly made of silty clays and plastic clays that are abundant at the City Centre, Dagoretti, Karen, Bernhard, Kasarani, Garden, Madaraka, Adams Arcade, Woodley, Jamhuri , Marurui and Nairobi South Estates. The thicknesses range between 2 and 22 m. The thickest deposits are found at the Lutheran Church, Boulevard Hotel, Norfolk Hotel and Kijabe Street. These soils are saturated at most localities with very unusual Atterberg limits and are highly compressible. Design pressures on these soils have been in the range of 75-180 kN/m². Distress in structures supported on these soils is reported in Garden Estate, Karen, Madaraka and Nairobi South. Construction records indicate that the alluvium is also liable to collapse in excavations.

Character of Karura, Limuru and Kabete Trachytes

These are very competent rocks when fresh; they are generally fine-grained and weather to red brown soils or plastic clays with low density and sensitivity to moisture. The plasticity of soils weathered from these rocks is higher when the soils occur in a valley. There is caving in boreholes drilled through Kabete and Karura trachyte soils. The soils also subject to burrows and sinkholes that allow seepage of water into foundation bases leading to distress in structures.

Character of Kerichwa Valley Series

Kerichwa Valley Series is stratified in thin alternating weak and strong layers of various shades; the rocks have low to very low UCS. The allowable pressures on KVS are as follows: for intact yellow tuff- 500 kN/m^2; black tuff -1600 kN/m^2; fractured tuff -350-500 kN/m^2; weathered tuff 250-350 kN/m^2, and decomposed tuff- 0-200 kN/m^2. Local pockets of decomposed tuff are encountered randomly across construction sites indicating fracturing. The black tuff is considered to provide rafting effect on the plastic clay underneath it when it supports structures. The yellow tuff expands when in contact with water while the tuff matrix in agglomeratic tuff is very sensitive to drilling or excavation through Nairobi phonolite.

Character of Nairobi Phonolite and Nairobi Trachyte

The Nairobi Phonolite occurs in more than four layers, some intercalated with agglomeratic tuff. The rock is encountered at 14 m to 18 m depths and has moderate to high strength. The allowable pressures on Nairobi Phonolite range between 4467 kN/m^2 to 29 463 kN/m^2 and is therefore the most competent rock in the study area.

Nairobi Trachyte weathers to reddish brown plastic soil with very little gravel content. The soil settles when loaded and collapses when flooded. Most distressed structures in Karen and Langata are located on the derived soils. At community, the rock as few spots of low and very low UCS values. For instance, it has allowable pressure of 409 kN/m^2 at 3.85 m depth at PTA Plaza at Upper Hill while at 8 m depth at NCPD it has allowable pressure of 11673 kN/m^2. From test results, it can safely carry 1500 kN/m^2 when fractured and 3000 kN/m^2 when fresh. The weak spots in the Nairobi Trachyte could be related to fracture traces.

6.1.5 Subsoil liable to collapse and effects of skyscrapers

Weathered/decomposed tuffs, agglomeratic tuffs, silty clays and red clays collapse in excavations when left open. Shoring and/or underpinning operations should be considered when excavating adjacent to existing foundations especially those supported on shallow foundations. This is especially so when the excavations will remain open for a long time. When an excavation is likely to encounter the subsoil mentioned above, shoring systems should be included in the design.

At the City Centre, settlement due to the weight of individual sky scrapers or interference settlement can only affect layers extending to 30 m depth. Within this depth are intact layers of tuff and phonolite that cumulatively are within a range of 12 to 16 m thickness. Real-time settlement measurements indicate that the weak layers do not only settle but they also heave. The settlement and heave magnitudes are within the range of 13 to 18 mm for well constructed structures. Most types of structures tolerate such magnitudes (Fenton *et al.*, 2003).

6.1.6 Causes of distress in structures and remedial measures

The distresses in the structures investigated in this study occurred due to any of the following: fractures covered with alluvium, swamp and clay soils; moisture sensitive soils; expansion of yellow tuff/ agglomeratic tuffs in contact with water; mistakening of decomposed agglomeratic yellow tuff (gravelly clay) for laterite and designing for laterite thus overloading foundation soils; laterites of variable profile and bearing capacity underlain by saturated soft soils within construction sites; and, seepage of water into burrows and sinkholes in red clays.

Distress is most common in structures supported on erratic soils, expansive clays and collapsible soils that are moisture sensitive. The moisture sensitivity in the soils varies considerably within layers and the ranges of swell pressures obtained are large. Swell pressure magnitudes of 156-209 kN/m^2 obtained at one site at Ngei Estate indicate that the expansive soils can cause severe distress in lightly loaded structures. The subsurface profiles indicate that there are many alternating thin layers of weathered and fresh materials up to about 30 m and thus taking foundations to deeper levels will not necessarily produce safer foundations. However, at 14 m depth at the City Centre, the materials are more uniform with depth.

Saggerson (1991) gives the following statement that is very important in this study "Elsewhere in the Kikuyu highlands, numerous sinkholes were noted in the extremely red soils overlying trachytes and tuffs. These hollows are not associated with faults but obviously represent subsidence within the soft, often incohesive soil after rain. As in the case of sinkholes at Muguga, underground cavities may exist where earth cracks cross pervious layers, thus permitting downward movement of soil."

Grading of most of the tested soils shows that they contain a substantial fraction of silt that is extremely moisture sensitive. Structures constructed in the vicinity of the alluvium, swamp soils and clays have also experienced distress showing that these sensitive soils are more extensive than indicated on the geological map. This is because discussion of the distribution of geologic materials and processes commonly involves a "megascopic" scale of several metres or kilometres while engineering properties are discussed in "microscopic" context.

From the development of distress in structures around Hazina Towers, it can be observed that factors controlling strength of subsoil around an excavation are complex. These involve amongst other things, the slope and depth of the cut and the length of the time the excavation is left unsupported or partially supported. Major damage can occur due to adjacent construction excavations, and even in cases of minor damage, the resulting insurance and legal issues can be expensive and distracting. This case also shows that the ground was slow to respond to movement and also demonstrates the importance of rapid provision of early positive supports in the form of struts or props if movements are to be kept at minimum. Moreover, it is evident that any attempts to estimate the magnitude of movements outside the excavations needs to take into account time required providing support.

Several methods of construction have been used successfully to prevent damages to lightly loaded structures supported on sensitive and variable soils in Nairobi. The methods of construction include:

i) Placement of hardcore or mass concrete before laying foundations on expansive soils;
ii) Use of strapped foundations on subsoil with variable profile;
iii) Development of proper drainage for surface and groundwater;

iv) Construction of apron around structures supported on collapsible and expansive soils;

v) Provision of moisture barriers around structures supported on collapsible/ expansive soils;

vi) Removal and recompaction of sensitive soil or replacement with imported suitable material, and;

vii) Inundation before foundation placement.

One problem of the inundation method of treatment is knowing the extent to which the water will be able to penetrate and the time it will take. Dawson (1959) flooded foundation trenches for a building on expansive clay. After a four month period of flooding, the penetration of the water into the soil was found to be very limited. McDowell (1959) also mentions the use of surface flooding to heave highway formations but states that in one instance water contents had increased to a depth of only 1.2 m after 28 days of flooding. Recent studies have indicated that in situations like these, it is good to combine trenches with wells. The only problem with wells is that the percolation will continue after the main flooding process resulting from water left in the wells. The other problem with the method is that as the soil would have softened, a large superimposed loading may cause settlements as serious as the heave movement. For this reason, pre-heaving by flooding only appears to be a feasible technique for lightly loaded structures.

Another suitable method in such conditions is by use of floating foundation. For tall structures, the building should be placed with about quarter of its height under the ground surface to get full flotation (Hamsey, 2000). The basement can be left void and as parking not for storage. However the following questions have to be answered: Just how deep into the soil should the building be placed?; would the excavation have to be enclosed by a wall during construction to prevent cave-ins of the soil?; would it be necessary to lower the water table in order to excavate and construct the foundation, and, if so, what means should be used to accomplish this lowering of the groundwater?; is there a danger of damage to adjacent buildings?; and, how much would the completed building settle and would it settle uniformly?(Kany, 1977)

6.2 Contribution to knowledge

6.2.1 Groundwater rest level variations

This study presents results of an investigation on temporal and spatial groundwater rest level variations that indicates that the rest levels have been receding due to exploitation. If the trend continues in response to increase in population and increasing industrial activities, the aquifers could be depleted. This depletion could lead to long-term surface subsidence especially at localities with concealed faults. The study therefore provides a basis for establishing a reliable hydrogeological and scientific framework for monitoring and determining suitable strategies for groundwater augmentation so that subsidence does not occur.

6.2.2 Fault traces

This study has traced faults in boreholes to determine their possible association to sudden change in subsurface profiles, borehole yields, and structural defects. The nature of fault traces in wells indicates that several faults could be existing underground without being expressed on the surface. The fault traces are shallower in the west where some can be observed on the surface but deeper eastwards. Existence of such fractures beneath Civil Engineering structures cannot be ruled out. Due to the proximity of the City to L. Magadi and L. Natron in the Rift Valley where earthquake activity is high, thorough geotechnical investigations should be carried out before construction of heavy structures in areas with soft subsoil.

6.2.3 Character of the subsurface materials

This study has analysed past geotechnical investigation results that have hitherto been unavailable for use in design and provides the engineering properties and expected performance of the various subsurface materials encountered. The results are presented in tables and figures that are easy to comprehend by consultants and developers in the city. It has also analysed borehole logs for water supply wells and provides profiles that bridge that gap in knowledge about the nature of subsurface beyond that encountered in geotechnical boreholes and construction excavations.

6.2.4 Causes of distress in structures.

This study has found out that there are several causes of distress in structures apart from the erratic soils, expansive clays and collapsible soils. In Onyonka and Ngei estates as well as Mathare, lateritic clays are mistaken for laterites and designed to support structures using the bearing capacity of laterites resulting in overloading of the foundation soils. Some structures in Karen, Langata and Mbagathi have undergone distress because they are located on faults with deep unconsolidated soils. Underpinning of structures located on the faults by deepening of foundations has resulted in new cycles of settlement. Other causes of distress include settlement of compressible or improperly compacted fills, overloading of deep soft soils, sloping formations as well as improper maintenance around foundations.

6.2.5 Strengthening past reports

A fault trace from Industrial Area emerges through Moi Avenue at the city centre and affects structures as far north as Muthaiga. The subsurface profiles along Moi Avenue are different from those from other parts of the City Centre. For instance the brown agglomeratic tuff is only encountered along Moi Avenue and Uhuru Highway. This study therefore strengthens the observations by earlier researchers that before the formation of the Kerichwa Valley Series rocks, the course of Nairobi River was along Moi Avenue towards Railway Station.

6.2.6 Papers published in International Journals

1. Defects in structures in Nairobi City: Causes and mitigative strategies, International Journal of Disaster Management and Risk Mitigation, Vol. 3, 2009.
2. A study on the engineering behaviour of Nairobi subsoil, APRN Journal of Engineering and Applied Sciences Vol. 6 No. 7, July 2011.
3. Dealing with sensitive and variable soils in Nairobi City, International Journal of Research and Reviews in Applied Sciences Vol. 19 Issue 2, Nov 2011.

6.2.7 Papers published at dissemination forums

1. Engineering behaviour of Nairobi subsoil, Institution of Engineers of Kenya Conference, 11th -13th May 2011, Nairobi, Kenya.

2. Effects of drilling deep tube wells in the urban areas of Nairobi City, presented at the International Conference of Earth Sciences and Engineering, World Academy of Science, Engineering and Technology (WASET), Nardeen Netherlands, 2011. Under review at the International Journal of Earth Sciences and Engineering.
3. Spatial modelling of groundwater conditions in Nairobi City using Geographic Information System, Presented at the 15th International Conference for Women Engineers and Scientists, 19-22 July 2011, Adelaide, Australia.
   Under review at the Nile basin Water Science and Engineering Journal.
4. Variation of Groundwater static levels in Nairobi City since 1927 and its social implications, Biannual Conference by the Geological Society of Kenya, March 21-23 2012.

6.3 Recommendations

Below is a summary of the recommendations made regarding each of the specific objectives.

6.3.1 Rest level variations and probable settlement

The largest percentage of the drop in groundwater rest levels has occurred in the last two decades that have contributed a total of more than 1000 new wells. There is need for a research on the possibility for groundwater augmentation by artificial recharge. Artificial recharge schemes have proved to be a remedial option for controlling groundwater drawdown and ground subsidence in some parts of the world (Newton, 1981). Drilling of monitoring boreholes in the vicinity of meteorological stations to evaluate natural recharge and determine safe yield is also recommended. So far the safe yields are based on results of pumping tests which are not necessarily reflective on the amount of water available in the long term.

6.3.2 Subsurface profile and effects of groundwater abstraction

Surface subsidence due to groundwater abstraction has not been observed and there are no defects in structures that can be related to drawdown. This is because no monitoring has been carried out at the ground surface nearby the distressed structures. It would therefore be worthwhile to install a few real-time settlement metres (extensometers) at sites with high yield boreholes such as Industrial Area, and at areas with potential of large settlements such as Kiserian and Kileleshwa. So far, it is assumed that no subsidence can occur because of the thick

Nairobi Trachyte and Nairobi Phonolite layers shielding the ground surface from any consolidation within the Athi Series aquifers and aquitards.

6.3.3 Variable subsoil

Variable profiles and weak spots are encountered in many parts of the study area. Before construction of any structure, it is worthwhile to make a detailed study at the site, to locate stronger and weaker zones and to investigate comprehensively the relation between total settlement and differential settlement. 2-D and 3-D tomography should be adopted for routine site investigation because it tends to give continuous and detailed information compared to that derived from drilling.

The presence of concealed fracture traces in the study area calls for a more precise method of delineating them. Two-dimensional tomography and magnetic methods can be used in such as study. This would enable identification of the weak areas that support structures and are subject to seismic effects. Better engineered structures would then be recommended for such areas.

6.3.4 Engineering properties, foundation design and structural distress

Geotechnical investigations are recommended for areas prone to distress in structures and in the vicinity of alluvium, clays and swamp soils. At such sites, the successful construction methods discussed here can be used to mitigate distress in structures. If it is not possible to carry out a site investigation, results of this study can be used for design and the rest of the details can be obtained during construction. Satisfactory foundation performance includes consideration of numerous factors in addition to the indicated allowable bearing values. These factors include settlement performance, general stability and effects of and on adjacent manmade or natural features.

Whereas it is possible to design buildings to accommodate settlements due to their own weight, hogging often occurs in existing buildings as a result of nearby construction. It is therefore essential to choose a suitable foundation type, construction sequence and adequate monitoring program to control deformations of existing structures. The fast-paced construction and excavation of foundation bases at alternate positions controlled ground movement and possible distress of buildings adjacent to I&M building that was constructed at the same time with the Hazina Towers

6.3.5 Weak subsoil and shoring and/or underpinning of foundations

Weathered / agglomeratic tuff, Kabete Red Clay and alluvium collapse in excavations. When making excavation in this subsoil, it is advisable that the contractor in liaison with the consulting engineer do all within their control to help prevent damages from occurring on adjacent structures. This can be done by timely shoring of excavation sides when the foundations of the adjacent footings are not exposed or construction of diaphragm walls before the excavation begins. Shoring of old low-rise buildings is highly recommended because most of them are supported on laterite or weathered rock beneath which is decomposed yellow tuff or plastic clay.

Most of the underpinning cases described in this study were successful and can thus be replicated at sites with similar subsoil conditions. The unsuccessful cases in Karen and Langata areas were located on faults and the thus underpinning initiated new cycles of settlement. Before underpinning any structure in Karen and Langata area, the contractor should ascertain that it is not located on/or nearby a fault by reading the geological reports by Saggerson (1991) and Gevaerts (1964) in addition to obtaining logs for the adjacent water supply boreholes. The best methods of underpinning structures in Karen and Langata area would either involve widening of the foundation bases to spread the pressure over a wider area or by treating the soil beneath the foundation. The same underpinning methods should apply to distressed structures on Kabete Red Clays because these soils can be deep (60 m), according to Saggerson (1991).

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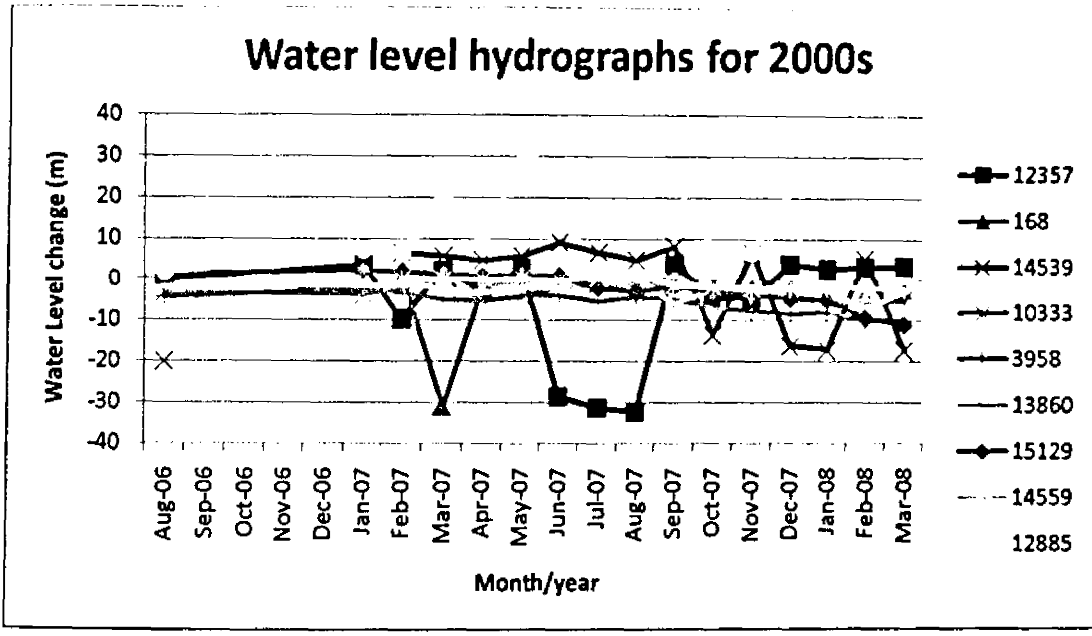
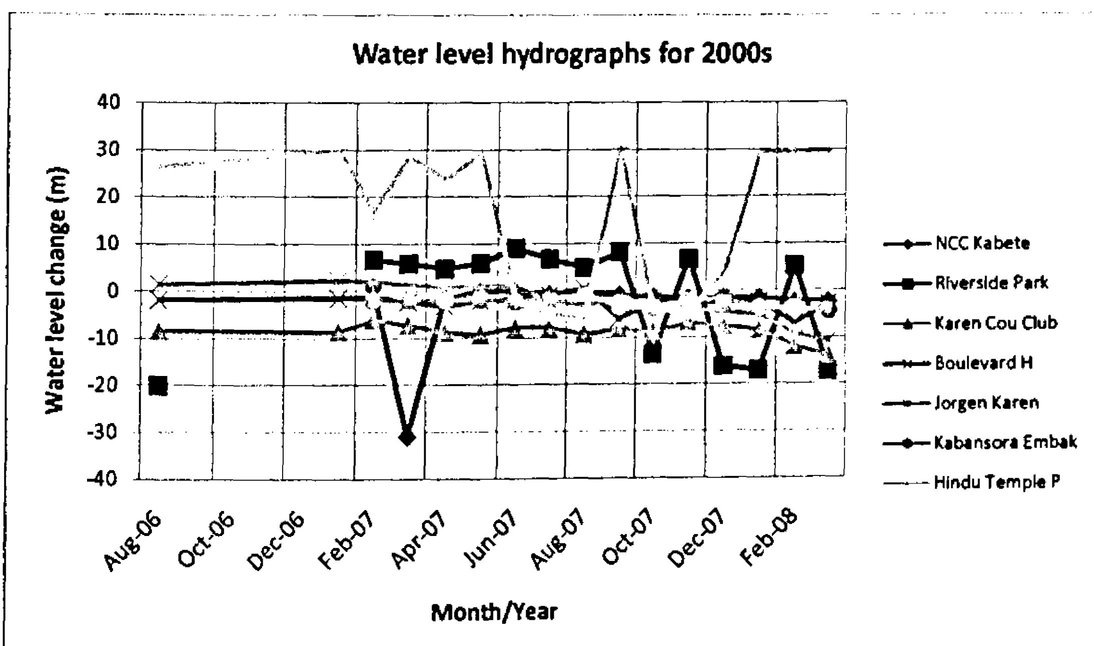
http://www.therixgroup.com.au/underpinning 12 June 2012

Appendix 1

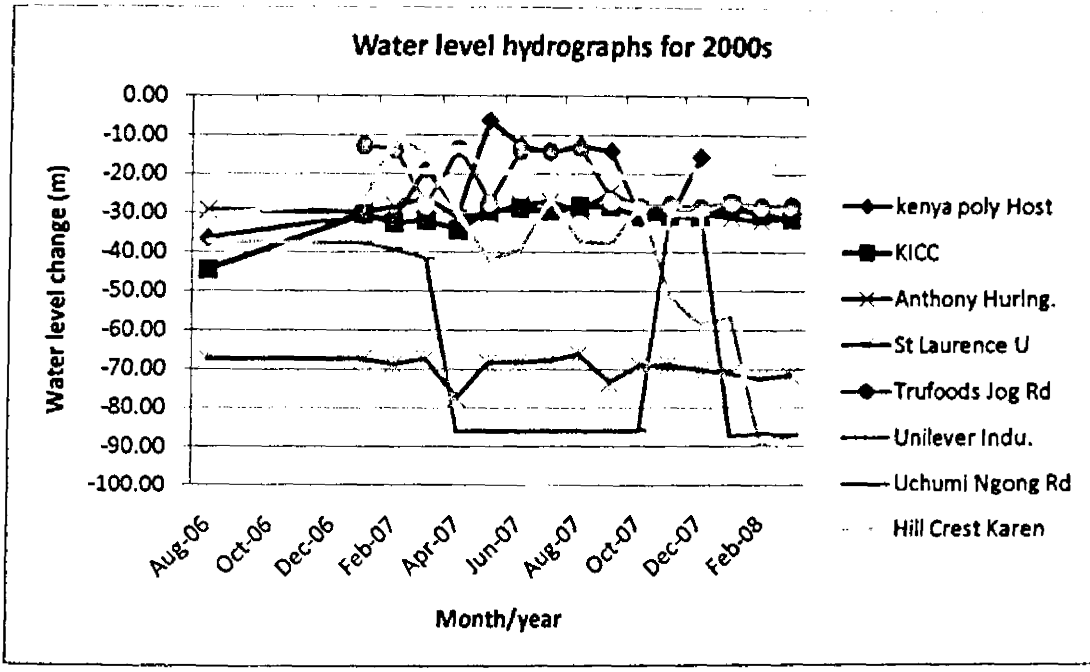
Water level hydrographs for some monitoring wells



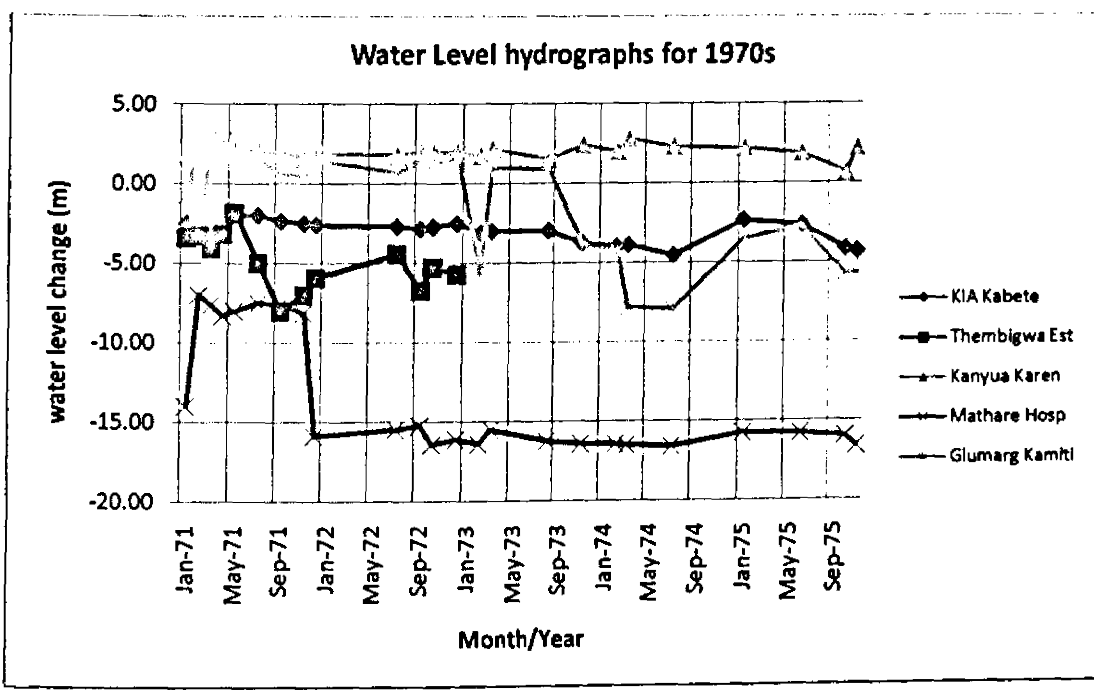
Appendix 1 (a)



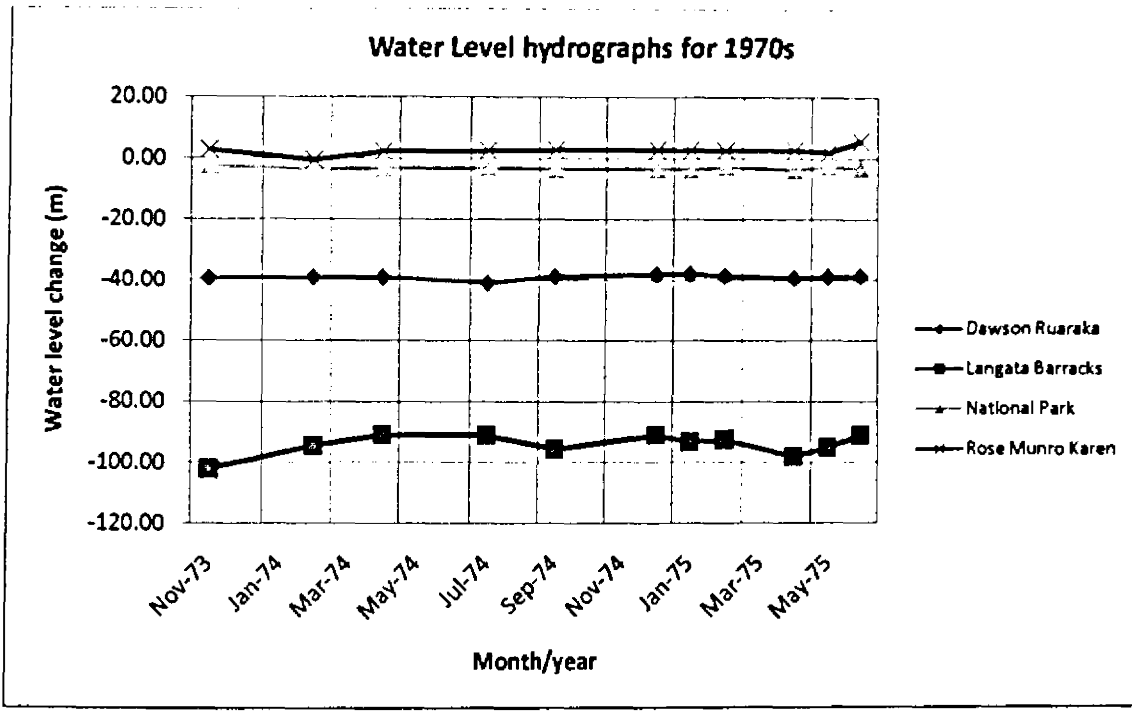
Appendix 1 (b)



Appendix 1 (c)



Appendix 1 (d)



Water Level hydrographs for 1970s
20.00
0.00
-20.00
-40.00
-60.00
-80.00
-100.00
-120.00
Water level change (m)
Nov-73
Jan-74
Mar-74
May-74
Jul-74
Sep-74
Nov-74
Jan-75
Mar-75
May-75
Month/year
Dawson Ruaraka
Langata Barracks
National Park
Rose Munro Karen

Appendix 1 (e)

Appendix 2

Summary of geoelectric properties obtained from Interpex sounding curves

Appendix 2: Summary of geoelectric properties obtained from Interpex sounding curves

Site	Layer Resistivity (ohm-m)						Layer Thickness (m)				
	1	2	3	4	5	6	1	2	3	4	5
Kasarani Compex	604	42	557	59	557	77	1.4	3.9	12.5	35	12
Bulbul Fault	132	3.6	20	38	19	450	1.2	2.6	9.9	37.9	324.3
Bulbul KMC	2688.3	219.5	44.2	79.7	5.7		0.4	1.0	45.5	105.3	
CUEA	9.3	2.7	20.3	23	13.2		0.1	2.2	65	400	
City Park	764	63.7	102.9	65.8	14.9		0.53	2.9	34.95	77.1	
Dagoretti Approved	34	166.3	27.7	1930			0.53	4	35		
Dagoretti High	290	53.8	27.2	103	41.7		0.2	5	19	78	
East gate	15.9	73.9	275.4	42.4			1.5	14	37		
Eastleigh High	8.8	84	148	227			2.3	13.2	109		
Edevale Home	11.8	784	61	295	33.3		1.4	4.8	17	84.8	
Gacharage Primary	14.8	92.4	98.5	34	253		0.14	8.4	16.9	50	
Garden Estate	6.7	32	58	53	159		0.15	11.2	41.3	121	
Gathiga Primary	377	100	167	32	54.5	125	0.7	1.26	6.7	39.5	40
Kabete Approved	380	41.7	66	20	136		0.18	5.5	4.8	59	
Kahuho High	80	19	179	43.6			0.27	2.1	8.3		
Kaloleni Primary	11.4	20.2	186	4.7			6.13	8	38.6		
Kanjeru Primary	144	312	100.5	110.9	532		0.44	1.5	12.4	140	
Karen college	288	29.2	76.4	244.2			0.44	7.5	350		
KISM	870	52	73	37.6	15.3		2.1	10.2	29.2	25.1	
Kibiku Primary	375.7	79.6	127.8	52.6	129.26		0.2	7.4	12.6	75.1	
Kihara Secondary	77.9	133.3	38.1	5.9			1.6	2.2	70.3		
Kikuyu Day Sec.	505.6	50.8	122.5	533.2			1.6	9.5	56		
Ruthimitu Girls	417	27.9	110.4	20.4	72.7	18.2	1.8	3.7	7.2	23	87
Kwarara Rd	98.2	35.5	212.6	77.2	223.5		1.2	4.5	47.6	172	
Lenana School	7.4	2.5	271.1	46.9	18		1	1.2	10.7	177	
Lower Kabete	244.5	915.6	171.9	32.8	43.7		0.6	2.26	7.3	81.7	
Makadara Railway	3.8	20.5	62.6	35.1			0.6	9	26		

Site	Layer Resistivity (ohm-m)						Layer Thickness (m)				
	1	2	3	4	5	6	1	2	3	4	5
Miremma Drive	232	17.7	231.8	4.6	18.3		2.5	4.2	9.2	101.4	
Mutuini Primary	48.9	31.2	45.6	41.2	54.6		1.5	8.6	15.8	124.2	
Nairobi Tech	16.9	35.4	131.5	42.7	94.1		1.4	1.9	4.6	17.1	
Ndege Primary	5.1	51.6	684	3.3			0.5	9	47.1		
Ndurarua Primary	382.2	156.3	77.5	500	176		0.24	5.9	57.2	66.3	
Nembu Primary	20.8	111.3	30.3	88.4			0.5	6.2	18.9		
North Airport Rd	10.7	72.5	340.8	34.3			0,2	16	82		
Bomas of Kenya	5.5	21.4	178.8	6.1			1.0	4.5	134		
Othaya Rd	104	13.5	61.9	9.5			1.8	11.4	25.5		
Pepo Rd	477.7	26.1	220.61	23.9	919.5		1.83	7.6	10	39.3	
Pumwani Sec	20.9	48.1	124.3	32.5	273.5		1.35	1.6	8.6	37	
Ruthimitu Girls	17.8	57.7	25.2	238.7			0.35	5.3	34		
Austins Bernhard	163.5	23.4	331.9	28.2	44.8		2.0	7.9	41.4	282.7	
Uhuru Gardens	17.5	44.9	75.2	434.7	63.8		1.9	3.6	57	192	
Kirandini Telkom F	5.6	169.9	55	106.6			0.7	7.5	172.5		
Uhuru Park	27	15	8	2400			1.5	2.5	36	103	
Nairobi University	47	10.1	135.5	109.1	8.5		1.64	19.9	39.2	34.5	
Upper Kabete	371.4	171.31	643	42.8	482.9		144.6	2.6	2.4	82	
Upper Kabete L	121	24	109.3	30.3	135.1		0.3	1.8	2.5	183	
Waiyaki Way	109.8	17.4	264.8	11.4			0.5	14	54		

Appendix 3

Transmissivity and storativity values calculated for some boreholes

Appendix 3: Transmissivity and storativity values calculated for some boreholes

BH ID	LOCALITY	GRIDX	GRIDY	TDEPTH	WRL	YIELD	DRAWDOWN	RECOVERY	TRANSM	STORATIV
3683	Langata	36.733	-1.335	153	85	7.62			0.00126	0.00664
3731	Miotoni Karen	36.750	-1.316	305	34	6.78			0.00121	0.00744
3815	Eastleigh	36.850	-1.266	152	8	6.78	13.4	2	0.00103	0.00602
3909	Karura Forest	36.832	-1.248	152	102	0.48	15.2	4	0.00233	0.01
4140	Garden Estate	36.866	-1.216	163	95	7.38	28	12.3	0.0115	0.0198
4148	Kamiti	36.716	-1.333	185	105	5.22	46.4	41	0.00126	0.00904
4190	Karen	36.694	-1.348	143	24	2.88	39.1	26	0.000282	0.00204
4266	Lower Kabete near KIA	36.745	-1.237	61	2	6.96	6.2	2	0.0214	
4268	Kitisuru	36.783	-1.235	218	52	5.4	62.9	32	0.000847	0.00457
4277	Mwimuto peponi rd	36.775	-1.229	206	42	10.8	38.1	32	0.00366	0.0139
4419	Kasarani	36.895	-1.217	116	21	17.5	4	1	0.0357	0.0562
4438	Gathiga	36.774	-1.229	150	44	16.2	61		0.000988	0.00445
4446	National Park	36.760	-1.393	200	49	6.6	12		0.00358	0.0254
4459	DANDORA	36.900	-1.257	154	34	27.6	7	3	0.0306	0.0113
4466	Red Hill Road	36.763	-1.226	200	41	6.6	13		0.00381	0.0259
4520	Jaydes knitting Lunga Lunga	36.857	-1.302	180	87	10.9	35	2	0.000181	0.00159
4521	Karen	36.691	-1.317	170	10	1.2	74	4	0.000431	0.00248
4538	Kabete	36.742	-1.254	160		7.2	8	1	0.00171	0.00914
4580	Karen-Langata	36.666	-1.300	156	26	4.68	41		0.00314	0.0301
4599	Mbagathi Ridge	36.749	-1.375	144	19	17.3	34		0.00353	0.0354
4617	Kingeero	36.733	-1.252	200	15	5.58	64	3	0.000473	0.00331
4623	Embakasi	36.900	-1.283	202	12	24.7	58	8	0.000754	0.0073
4646	Lenana School	36.728	-1.297	150	78	6.78	26	10	0.000874	0.00456
4666	Woodley	36.766	-1.316	306	108	6.78	97	6	0.000208	0.00126
4714	Dandora	36.895	-1.261	80	38				0.00454	0.0197
4732	KCC Dandora	36.887	-1.270	196	76	22.5	15		0.00243	0.00126
4735	Judges, Ndege Road	36.721	-1.353	106	4	9.12	7	1	0.011	0.0494
4758	Langata Armed Forces	36.738	-1.340	312	44	1.8	9	6	0.000978	0.00558
4762	Chriss Harris Karura Ward	36.889	-1.272	247	71	11.7	72	3	0.000185	0.00155
4768	Karen Water supply Gitiba Karen	36.674	-1.299	117	4	19.8	40	12	0.00293	0.0131
4791	Upper Hill	36.804	-1.282	284	145	5.76	6	8	0.00813	0.0549
4823	Industrial Area	36.840	-1.299	203	15	13.6	95		0.00237	0.0104
4840	Bulbul	36.694	-1.338	224	38	4.08	47	8	0.000528	0.00215

Appendix 3: Transmissivity and storativity values calculated for some boreholes

BH ID	LOCALITY	GRIDX	GRIDY	TDEPTH	WRL	YIELD	DRAWDOWN	RECOVERY	TRANSM	STORATIV
4841	Karen	36.693	-1.334	160	94	9	38		0.0237	0.122
4846	Ngong	36.694	-1.336	148	66	7.2	1	8	0.11	0.297
4847	Ruaraka	36.905	-1.251	132	74	7.92	5	19	0.00811	0.0426
4860	Nairobi Steam Lau &dry cleaners	36.833	-1.306	200	37	21.3	40		0.00244	0.0103
4862		36.770	-1.392	176	85	5.4	3	14	0.00114	0.00894
4882		36.750	-1.333	150	89	10	34	1	0.000126	0.000638
4898	Unga Ltd Dakar Rd Industrial Are	36.834	-1.299	200	17	18.4	98		0.00307	0.0121
4901	Mutuini	36.695	-1.315	190	69	2.58	99	8	0.000126	0.000703
4902	Industrial Area	36.836	-1.293	150	30	1.92	29	8	0.000879	0.00609
4922	Dam Estate	36.777	-1.325	290	18	5.16	44		0.000787	0.00407
4923	Westlands	36.778	-1.254	180	50	15	56	2	0.002	0.00825
4925	Unga Ltd Lusaka Rd Rd Industrial	36.840	-1.300	200	15	17.2	97		0.00204	0.00986
4926	Kenya Science Teachers College	36.762	-1.298	204	111	4.08	44	8	0.000747	0.00403
4941		36.703	-1.333	210	95	10.2	24		0.00119	0.00611
4962	Langata	36.750	-1.338	150	102	4.5	10	1	0.00137	0.0071
5015	Industrial Area	36.854	-1.300	220	50	24	49		0.00434	0.0192
5026	Industrial Area	36.850	-1.304	220	74	19.5	42	8	0.00102	0.00531
5027	Industrial Area	36.861	-1.303	220		13	52	1	0.00497	0.00979
5041	Industrial Area	36.845	-1.307	200	8	12	129	6	0.00133	0.00653
5050	Aga Khan Hospital Private Wing	36.824	-1.259	171	21	9	71	1	0.00244	0.0107
5114	Industrial Area	36.852	-1.299	220	56	6.06	26	24	0.00102	0.00583
5174	Industrial Area	36.850	-1.306	200	27	6.66	13	8	0.0183	0.0683
5204	Miotoni Karen	36.727	-1.303	118	35	3.6	50		0.000399	0.00305
5205	Wire Products Indu. Area	36.849	-1.304	220	14	9.48	3		0.0267	0.133
5209	Gikabu Co Kariobangi	36.884	-1.251	145	59	16.3	27	5	0.00421	0.0124
5264	Ruaraka	36.883	-1.235	214	68	28	18	8	0.0191	0.103
5265	Kenya Breweries Ruaraka	36.883	-1.233	220	75	9	10	8	0.0118	0.0579
5342	Karen	36.716	-1.333	180	103	4.68	48	8	0.000455	0.0027
5351	Nyari	36.825	-1.191	179	106	3.78	53	12	0.000538	0.0027
5357	Thompson	36.753	-1.297	100	10	2.46	10	2	0.00101	0.0049
5365	Embakasi	36.900	-1.266	200	77	7.2	23	3	0.00439	0.0178
5399	National Park	36.833	-1.350	195	98	13	10	1	0.00532	0.0224
5406	Garden Estate	36.841	-1.220	220	92	7.98	28	8	0.00314	0.00988

Appendix 3: Transmissivity and storativity values calculated for some boreholes

BH ID	LOCALITY	GRIDX	GRIDY	TDEPTH	WRL	YIELD	DRAWDOWN	RECOVERY	TRANSM	STORATIV
5517	Footbridge, buruburu mathare	36.874	-1.264	182	79	8.16	76		0.00187	0.0157
5518	Golf Course	36.805	-1.300	278	149	7.68	14		0.00327	0.0123
5562	Gitiba	36.683	-1.316	150	65	6	10		0.013	0.0418
5676	Unilever Commercial St	36.832	-1.293	202	12	20.2	3	3	0.0495	0.214
5680	Orthodox Mission Kawangare	36.706	-1.221	180	102	4.02	13	1	0.00113	0.00639
6058	ST.BENEDICT MONASTRY	36.884	-1.228	180	60	12	6		0.083	0.42
6086	Dagoretti	36.700	-1.266	96	26	9.42			0.00111	0.00627
6223	Gitaru	36.672	-1.202	250	49	11.5	144	15	0.000054	0.000396
6224	Karen	36.723	-1.356	120	23	2.94	78	9	0.000117	0.000875
6225		36.750	-1.333	180	47	3.72	7		0.000539	0.00298
6228	Makadara	36.866	-1.300	120	11	9.06	9	8	0.00444	0.0192
6347		36.750	-1.350	200	95	4.26	33	1	0.000327	0.00217
6352		36.733	-1.233	200	15	10.2	30	4	0.00196	0.0108
6361		36.850	-1.250	132	20	1.2	87	9	0.000159	0.000631
6519	MINERAL MINING	36.850	-1.300	200	49				0.000454	0.00291
7109	Ongata Rongai	36.753	-1.388	160	15	6.78	10		0.0156	0.105
7303		36.800	-1.266	162		3.54	13		0.00275	0.0133
8697	Ololua	36.674	-1.355	164	105	7.8	14	3	0.00153	0.00865
8999	Industrial Area	36.850	-1.300	193	125		8		0.00407	0.0140

Appendix 4

Summary of boreholes with fracture traces

Appendix 4 : Boreholes with fracture traces

BOREHOLE INFORMATION USED FOR FRACTURE TRACE ANALYSIS

BH ID	Owner/Location	X	Y	FRACTURE TRACE
13878	Karanja Kabage Karen	36.750	-1.346	increase in yield with deepening
11951	Isaiah Kiplagat Bogani Rd	36.718	-1.372	Disappearing drilling mud
11454	Pat Kariuki Nandi Rd Karen Shoping Centre	36.708	-1.313	fractured at 173-176, disappearing drilling mud
10664	Park Place Hotel Ongata	36.753	-1.353	caving boulders
10852	M.P.Shah Parklands	36.814	-1.253	caving boulders
3659	LR 2259/66 Bogani Rd	36.733	-1.353	caving boulders
3646	LR7135/13 Embakasi	36.900	-1.358	caving pebbles
3605	614/77 Karura Avenue Muthaiga	36.800	-1.233	caving boulders
2656	LR 119 21/1 Rosslyn Estate	36.783	-1.216	caving boulders
2356	Kenton Camp Kileleshwa	36.786	-1.277	caving that could not be controlled by cementing
2331	LR 3586/1 Langata	36.753	-1.330	caving formation
2302	LR 978/2 Firestone East Africa	36.866	-1.326	caving formation
11744	Muthaiga	36.802	-1.246	caving formation
12600	Metroplastics Industrial Area	36.881	-1.304	continuously collapsing from 112-134 m, no drawdown during test
11130	Greenfields Investments Ltd Mogadishu Rd	36.858	-1.304	falling pebbles
11079	Pendip Joint Venture Kikuyu	36.669	-1.233	falling pebbles that hindered drilling rate
12114	LR 1160/215 Andrew Ndegwa Karen	36.712	-1.357	drilling stopped because of caving at 129-150 m
12890	William Ruchu Garden Estate	36.864	-1.222	caving formation
12873	Henry Mnairobi Westlands	36.810	-1.262	Excessive caving below 210 m
4732	KCC Dandora	36.888	-1.269	Excessive caving at 126-148 m
4721	Duke of Manchester Karen	36.709	-1.321	Original water level dropped and never recovered
4709	Mabati Ltd Olkalau Rd Industrial Area	36.866	-1.300	water level drop from 24 to 96 m when depth 192 to 196 was drilled
4714	Karen Water Supply Karen	36.895	-1.261	caving formations up to 80 m
4768	Karen Water Supply Gitiba Karen	36.674	-1.299	problematic caving from surface to 68 m
4122	Mwitu Estate Borehole Karen	36.683	-1.316	chronic caving
2841	Homewood Karen	36.695	-1.348	caving at 27-42 m necessitating casing, surface seepage up to 18 m
3001	Nairobi National Park	36.775	-1.379	Caving at 65-73 m
3159	KCCT Mbagathi	36.766	-1.383	boulder bed up to 120 m
3259	Kinoo Sublocation Dagoretti	36.695	-1.248	caving formations at 165-185 m with a lot of water
13165	Saito Yuzuri Rose Avenue	36.784	-1.279	Caving surface formations
13273	Madhu Paper Industrial Area	36.869	-1.308	collapsing formation at 104-118 m making drilling difficult
[illegible]	[illegible] Wilson Kinoo Shopping Centre	36.691	-1.252	backfilled by 10 m because of collapse at 50 80 m

Appendix 4 : Boreholes with fracture traces

11610	MAE Properties Runda	36.834	-1.217	collapsing formation from the surface to 9 m
11603	Kapa Oil Refineries Industrial Area	36.891	-1.307	backfilled from 153 to 140 beause of caving commencing at 110 m
11991	Appollo G. Mburu Nyathuna	36.681	-1.186	collapsing formations from the surface to 41 m
11951	Isaiah Kiplagat Bogani Rd	36.718	-1.351	fracture at 187-190, borehole water disappearing at 256-260 m
11452	Juanco Investments Ngong	36.669	-1.344	falling boulders at 60-72 m and 82-86 m
11428	Premier Academy Parklands	36.827	-1.265	caving problems
13528	Jacob Maina Muguga	36.667	-1.193	loss of circulation at 46-88 m
10664	Park Place Hotel Ongata Rongai	36.764	-1.376	chronic caving at 104-128 m
10626	E.P Wanjau Kibiku Ngong	36.667	-1.324	loss of circulation at 11-13 m and 41-58 m
10265	Rosemay Mumbi Rosslyn	36.797	-1.215	collapsing at 104-105 m
14045	William W Ngugi Umoja Innercore	36.891	-1.279	Collapsing formation producing a lot of water
13916	Blackrose Apartments Milimani	36.771	-1.289	single yielding aquifer that raised the head of water by 93 m
12578	Continental Holdings Westlands	36.803	-1.261	very productive aquifer with water flowing at very high pressure
14377	Spin Knit Dairy Industrial Area	36.867	-1.303	high yielding, a drawdown of 5 m after pumping $24m^3$/hr for 24 hrs
14507	Allotrope Properties Karen	36.741	-1.374	collapsing from 150 m. Drilling had to be stopped
14670	Cosmos Industrial Area	36.868	1.319	pumped at $30m^3$/hr for 24 hours with a drawdown of only 40 m
5205	Wire Products Indu. Area	36.849	-1.304	drastic drop in water level
14207	Horticultral Crop Dev Auth. Embakasi	36.920	-1.330	depth of 145-153 high flow with a lot of presure
5204	Bee keeping station Lenana	36.727	-1.303	caving formation at 84-102 m
5049	Midco Textiles Indu. Area	36.833	-1.300	unstable formation below 74 m
5022	Emco Glass works Gilgil Rd Industrial Area	36.845	-1.301	caving problem at 70-100 m
5502	Kenyatta National Hospital	36.803	-1.298	steady increase in water with deepening from 192-270 m
5562	J Wariua Ngong Karen	36.683	-1.316	borehole backfilled from 150 to 136 m due to caving at 116 to 135 m
6223	Mwangaza Jesuit Spiritual Masai Lane Kare	36.722	-1.355	caving boulders around 68-100 m
6222	A.R Gregory Karen Langata	36.715	-1.342	collapsing badly from 76-124 m and had to be cemented many times
6086	P K Karanja Dagoretti	36.700	-1.266	formation below 60 m caving
6309	Little Sisters of Sacret Heart Banana Hill	36.765	-1.185	caving from 80-85 m
7293	Norwegian Community School Karen	36.750	-1.334	caving at 107 m
6954	Stephen Kositany Langata	36.686	-1.369	collapsing at 220 m, abrupt change in formation Poor yield, 1 m^3/hr
4735	Judges, Ndege Karen Water Supply	36.721	-1.353	Total loss of circulation at 46 m, casing inserted at 12 m
4898	Unga Ltd Dakar Rd Industrial Area	36.840	-1.299	formation at 180-200 m collapsing. Fast recovery after pumping
4860	Nairobi Steam Laundry &Dry Cleaners. Dar	36.833	-1.306	formations caving lined the entire depth of borehole with casing
4876	Houndsmere Development Muthaiga	36.834	-1.256	caving formations at 30 m and 110-150 m
4841	Marula Lane Karen	36.712	-1.334	caving around 32-34 m

Appendix 4 : Boreholes with fracture traces

4828	National Construction Corporation Dandora	36.889	-1.272	cased to 20 m to prevent caving and seepage
4823	Elliot's Bakeries Changamwe Rd	36.840	-1.299	caving formations throughout
4947	Lion's Club Nairobi Gacharage Primary Sch	36.767	-1.203	rock strata very fractured below 30 m
4959	Balla Lavington	36.775	-1.275	caving at 12-24 and 36-38 m
4927	Tree Lane Karen	36.691	-1.311	hole 1 crooked, hole 2 broke the drillbit the third 180 m away okay
4926	Kenya Science Teachers College	36.762	-1.298	instable formation from 132-150 m
4925	Unga Ltd Lusaka Rd Rd Industrial Area	36.840	-1.300	caving formations, quick recovery oh head
13231	African Variety Incorporated	36.758	-1.360	minimum water drilled beyond recommended depth of 250 m
10082	Ngong' Children's Home Near KCB School	36.695	-1.340	Rock slumping caused change in diameter of well
10079	K.K Holdings Muthaiga	36.825	-1.251	caving at 120 m
10056	KCB Training School	36.698	-1.339	fracture at 176-178 caused water level to drop from 33 to 63 m
10060	LR 214/605 Muthaiga	36.817	-1.244	caving formation from surface to 35 m
10498	UoN Halls of Residence	36.812	-1.274	collapsing at fractured zones at 30-70 m
11045	Vincent Kambo, Karen	36.685	-1.332	there was a lot of caving
11019	Kangemi High school	36.748	-1.268	on penetration of lower aquifer, an artesian well was struck
11880	Highland Canners, Baba Dogo	36.881	-1.243	collapsing from 186 to 220 m
4790	State House BH1	36.805	-1.278	sudden hard formation broke a drillbit
4791	state House	36.804	-1.282	backfilled from 284 to 272 m, wet through out
11865	Dr Gikonyo, Karen	36.725	-1.331	Caving at 65-95 mwater level drop from 2 to 85 m
11704	Sameer Industrial Park, Msa Rd	36.867	-1.325	collapsing from 108-120 m
10524	Githaiya Njagi Ololua Ridge	36.716	-1.333	borehole backfilled from 18 to 115 m
1396	Muthaiga Golf club	36.839	-1.257	backfilled from 115 to 118 m
1366	Parklands Sports Club	36.812	-1.264	Nairobi Phonolite dipping at 75°, hole crooked vertical, caving
1318	H.D Archer	36.770	-1.242	water dissapearing into a fissure at 37 m
3652	Firestone E.A Factory	36.874	-1.328	Caving difficulties, another hole abandoned
3996	Sweppes and Cardbury Indu. Area	36.845	-1.304	caving cobbles and boulders
4538	Kabete Vet Lab	36.742	-1.254	excessive caving along the hole
11642	Stephen Waruhiu Red Hill Tigoni	36.712	-1.176	collapsing at 144-150 m
13528	Muguga Market	36.667	-1.193	loss of circulation at 46-88 m, 90-162 m

Appendix 5

Research questionnaire

Appendix 5: Research questionnaire

The purpose of this questionnaire is to carry out a study on foundation conditions at building sites in the city of Nairobi and methods used to ensure safe working conditions and safety/serviceability of finished structure.

NAME OF BUILDIN G	**PART OF THE CITY**	**TYPE OF STRUCTURE** (e.g. commercial, domestic, warehouse, e.t.c.) , **STOREY HEIGHT**	**DEPTH OF FOUNDATION** (m)	**ANY FOUNDATION CONSTRUCTION PROBLEMS** (e.g. groundwater flow, caving sides, change in assumed subsurface conditions, erratic subsoil, e.t.c)	**REMEDIAL MEASURES** (e.g. installation of pumps, sealing of basements etc **Underpinning** by pit method, jet grouting, compaction grouting, piling, minipiers/minicaissons, e.t.c. **Shoring** by External props, horizontal ties, raking shores, frying shores, dead shores, e.t.c.)

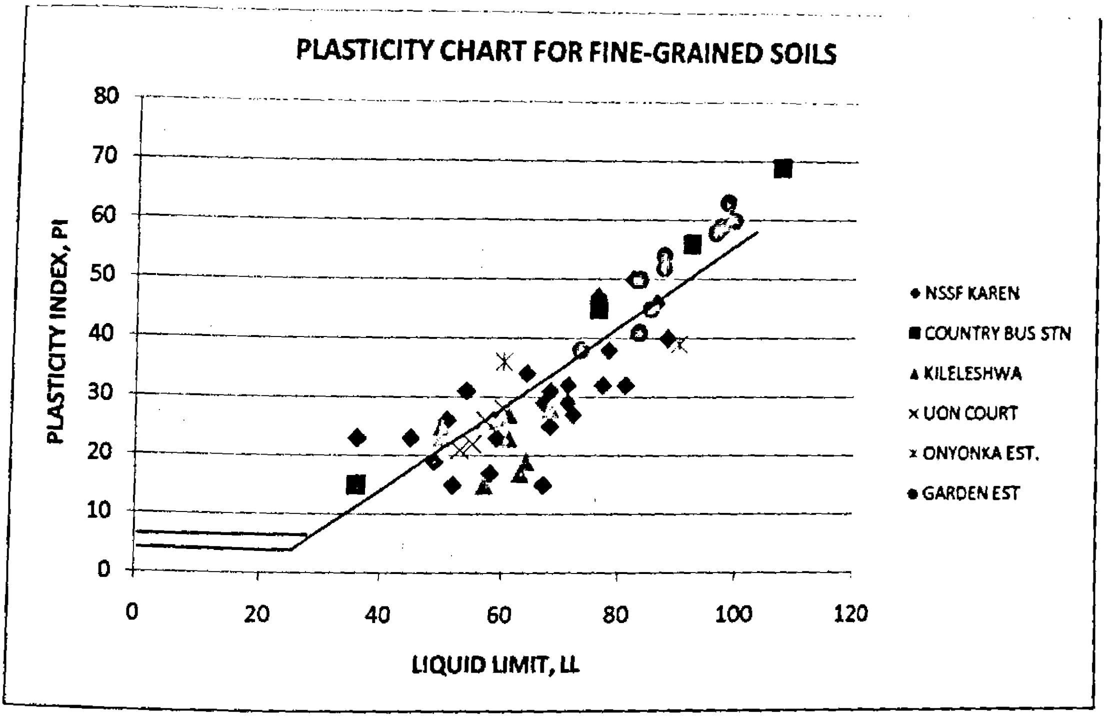
The purpose of this part of the questionnaire is to carry out a study on shoring and underpinning works that have been carried out in the City of Nairobi.

NAME OF BUILDING	**PART OF THE CITY**	**TYPE OF STRUCTURE** (e.g. commercial, domestic, warehouse etc) **, STOREY HEIGHT**	**DEPTH OF FOUNDATION (m)**	**REASON FOR SHORING** (e.g. weak adjacent foundation, weak subsoil, to underpin existing foundation e.t.c)	**METHOD OF SHORING** (e.g. External props, horizontal ties, raking shores, frying shores, dead shores, e.t.c.)	**LEVEL OF SUCCESS (scale of 1-5)** 1. barely succeeded 5 highly successful

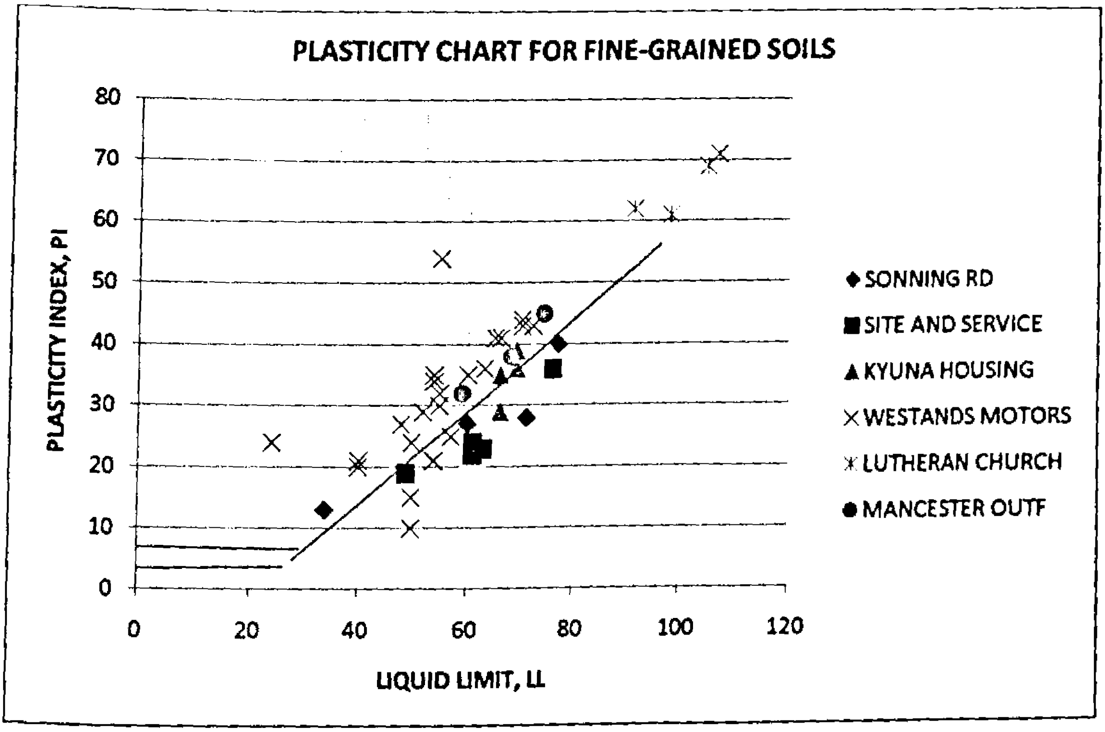
Appendix 6

Plasticity charts for sites within Nairobi city

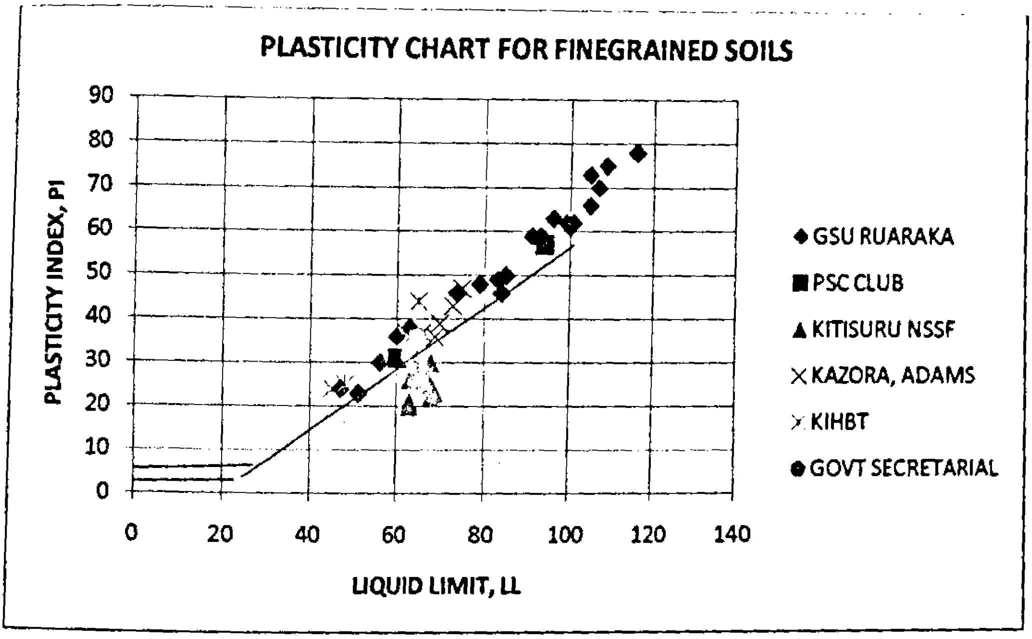
PLASTICITY CHARTS FOR SITES WITHIN NAIROBI CITY



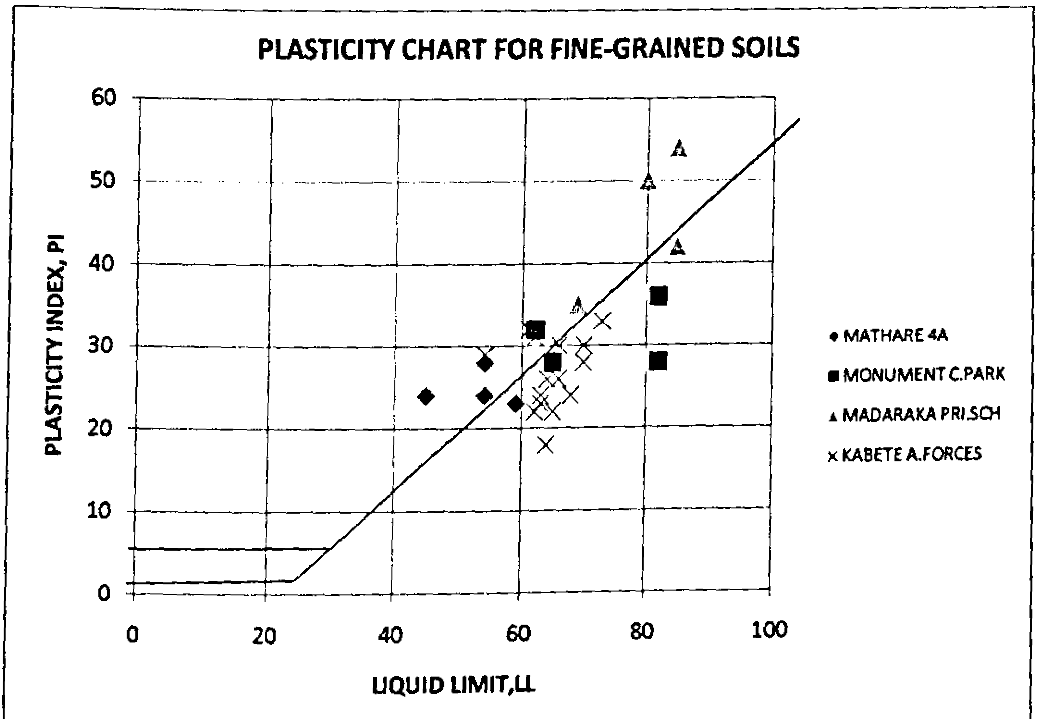
Appendix 6 (a)



Appendix 6 (b)



Appendix 6 (c)



Appendix 6 (d)

Appendix 7

VES results and the calculated UCS, ultimate and allowable bearing capacities

endix 7: VES results and the calculated UCS, ultimate and allowable bearing capacities

(m)	MN/2 (m)	ρ (ohm-m	logρ	logUCS	UCS(Mpa)	q_{ult} (kPa)	q_{all} (kPa)	τ_p (Mpa)
3: LENANA SCHOOL LAT 0117968 LONG 3643558 1000 FT BH								
1.6	0.5	6.06	0.78247	1.35649	22.725	61356	6136	-53.603
2.0	0.5	5.62	0.74974	1.34995	22.384	60438	6044	-54.243
2.5	0.5	5.51	0.74115	1.34823	22.296	60200	6020	-54.410
3.2	0.5	6.18	0.79099	1.35820	22.814	61597	6160	-53.436
4.0	0.5	7.18	0.85612	1.37122	23.508	63473	6347	-52.163
5.0	0.5	8.54	0.93146	1.38629	24.338	65714	6571	-50.690
6.3	0.5	10.70	1.02938	1.40588	25.461	68745	6874	-48.776
8.0	0.5	13.34	1.12516	1.42503	26.609	71845	7184	-46.903
10.0	0.5	16.02	1.20466	1.44093	27.601	74524	7452	-45.349
13.0	0.5	21.05	1.32325	1.46465	29.151	78707	7871	-43.030
16.0	0.5	25.23	1.40192	1.48038	30.226	81611	8161	-41.493
20.0	0.5	30.32	1.48173	1.49635	31.358	84666	8467	-39.932
25.0	0.5	36.23	1.55907	1.51181	32.495	87736	8774	-38.420
32.0	0.5	43.24	1.63589	1.52718	33.665	90895	9090	-36.918
32.0	10.0	43.83	1.64177	1.52835	33.756	91142	9114	-36.803
40.0	10.0	50.74	1.70535	1.54107	34.759	93850	9385	-35.560
50.0	10.0	60.24	1.77988	1.55598	35.973	97127	9713	-34.103
63.0	10.0	69.44	1.84161	1.56832	37.010	99928	9993	-32.897
80.0	10.0	70.09	1.84566	1.56913	37.079	100114	10011	-32.817
100.0	10.0	62.27	1.79428	1.55886	36.212	97773	9777	-33.822
130.0	10.0	57.13	1.75686	1.55137	35.594	96103	9610	-34.553
160.0	10.0	57.73	1.76140	1.55228	35.668	96304	9630	-34.465
200.0	10.0	49.69	1.69627	1.53925	34.614	93458	9346	-35.738
250.0	10.0	60.65	1.78283	1.55657	36.022	97259	9726	-34.046
250.0	25.0	55.92	1.74757	1.54951	35.442	95692	9569	-34.735
320.0	25.0	53.98	1.73223	1.54645	35.192	95019	9502	-35.035
400.0	25.0	31.27	1.49513	1.49903	31.552	85190	8519	-39.670
6-19: KIRANDINI TELCOM PLAYGROUND LAT 0118310 LONG 3644973								
1.6	0.5	7.84	0.89432	1.37886	23.926	64599	6460	-51.416
2.0	0.5	9.00	0.95424	1.39085	24.595	66407	6641	-50.245
2.5	0.5	10.44	1.01870	1.40374	25.336	68408	6841	-48.984
3.2	0.5	12.71	1.10415	1.42083	26.353	71153	7115	-47.314
4.0	0.5	15.60	1.19312	1.43862	27.455	74129	7413	-45.574
5.0	0.5	19.34	1.28646	1.45729	28.661	77385	7738	-43.750
6.3	0.5	23.64	1.37365	1.47473	29.835	80555	8056	-42.045
8.0	0.5	28.63	1.45682	1.49136	31.000	83700	8370	-40.419
10.0	0.5	34.66	1.53983	1.50797	32.208	86962	8696	-38.796
13.0	0.5	42.12	1.62449	1.52490	33.489	90419	9042	-37.141
16.0	0.5	48.50	1.68574	1.53715	34.447	93006	9301	-35.944
20.0	0.5	54.69	1.73791	1.54758	35.284	95268	9527	-34.924
25.0	0.5	61.34	1.78774	1.55755	36.103	97479	9748	-33.950
32.0	0.5	67.22	1.82750	1.56550	36.771	99280	9928	-33.172
32.0	10.0	62.50	1.79588	1.55918	36.239	97845	9785	-33.791
40.0	10.0	66.71	1.82419	1.56484	36.715	99129	9913	-33.237

Appendix 7: VES results and the calculated UCS, ultimate and allowable bearing capacities

50.0	10.0	69.58	1.84248	1.56850	37.025	99968	9997	-32.879
AB/2 (m)	MN/2 (m)	ρ (ohm-m	logρ	logUCS	UCS(Mpa)	q_{ult} (kPa)	q_{all} (kPa)	τ_f (Mpa)
63.0	10.0	68.73	1.83715	1.56743	36.934	99722	9972	-32.984
80.0	10.0	65.91	1.81895	1.56379	36.626	98890	9889	-33.340
100.0	10.0	64.76	1.81131	1.56226	36.497	98543	9854	-33.489
130.0	10.0	65.12	1.81371	1.56274	36.538	98652	9865	-33.442
160.0	10.0	66.81	1.82484	1.56497	36.726	99159	9916	-33.224
200.0	10.0	69.97	1.84491	1.56898	37.067	100080	10008	-32.832
250.0	10.0	71.22	1.85260	1.57052	37.198	100435	10043	-32.682
250.0	25.0	69.50	1.84198	1.56840	37.017	99945	9994	-32.889
320.0	25.0	66.93	1.82562	1.56512	36.739	99195	9919	-33.209

VS-23: CATHOLIC UNIVEVERSITY Lat 0120935 LONG 3644919

1.6	0.5	17.51	1.243286	1.448657	28.097	75861	7586	-44.594
2.0	0.5	15.71	1.196176	1.439235	27.494	74233	7423	-45.515
2.5	0.5	17.10	1.232996	1.446599	27.964	75503	7550	-44.795
3.2	0.5	19.58	1.291813	1.458363	28.732	77576	7758	-43.645
4.0	0.5	22.69	1.355834	1.471167	29.591	79897	7990	-42.393
5.0	0.5	25.83	1.412124	1.482425	30.369	81995	8200	-41.293
6.3	0.5	28.01	1.447313	1.489463	30.865	83335	8333	-40.605
8.0	0.5	31.92	1.504063	1.500813	31.682	85541	8554	-39.496
10.0	0.5	61.11	1.786112	1.557222	36.076	97406	9741	-33.982
13.0	0.5	40.05	1.602603	1.520521	33.153	89513	8951	-37.569
16.0	0.5	69.93	1.844664	1.568933	37.062	100068	10007	-32.837
20.0	0.5	51.31	1.710202	1.54204	34.837	94060	9406	-35.466
25.0	0.5	61.55	1.789228	1.557846	36.128	97546	9755	-33.921
32.0	0.5	74.87	1.874308	1.574862	37.572	101444	10144	-32.257
32.0	10.0	59.59	1.775173	1.555035	35.895	96917	9692	-34.195
40.0	10.0	66.00	1.819544	1.563909	36.636	98917	9892	-33.328
50.0	10.0	72.19	1.858477	1.571695	37.299	100707	10071	-32.567
63.0	10.0	79.79	1.901948	1.58039	38.053	102743	10274	-31.717
80.0	10.0	85.22	1.930552	1.58611	38.558	104106	10411	-31.158
100.0	10.0	88.99	1.949341	1.589868	38.893	105010	10501	-30.790
130.0	10.0	88.48	1.946845	1.589369	38.848	104890	10489	-30.839
160.0	10.0	85.62	1.932575	1.586515	38.594	104203	10420	-31.118
200.0	10.0	56.72	1.753736	1.550747	35.542	95965	9596	-34.614
250.0	10.0	95.49	1.979958	1.595992	39.445	106501	10650	-30.192
250.0	25.0	23.50	1.371068	1.474214	29.800	80460	8046	-42.096
320.0	25.0	30.60	1.485721	1.497144	31.416	84822	8482	-39.854
400.0	25.0	19.16	1.282396	1.456479	28.607	77240	7724	-43.829

VS-32: UHURU PARK , LAT 3649103, LONG 0117398

1.6	0.5	13.81	1.14019	1.42804	26.794	72344	7234	-46.609
2.0	0.5	5.86	0.76790	1.35358	22.572	60946	6095	-53.888
2.5	0.5	4.13	0.61595	1.32319	21.047	56827	5683	-56.858
3.2	0.5	3.78	0.57749	1.31550	20.678	55829	5583	-57.610
4.0	0.5	3.94	0.59550	1.31910	20.850	56294	5629	-57.258
5.0	0.5	4.47	0.65031	1.33006	21.383	57733	5773	-56.186

Appendix 7: VES results and the calculated UCS, ultimate and allowable bearing capacities

6.3	0.5	5.14	0.71096	1.34219	21.988	59369	5937	-55.001
8.0	0.5	5.78	0.76193	1.35239	22.511	60778	6078	-54.004
AB/2 (m)	MN/2 (m)	ρ (ohm-m	logρ	logUCS	UCS(Mpa)	q_{ult} (kPa)	q_{all} (kPa)	τ_f (Mpa)
10.0	0.5	6.56	0.81690	1.36338	23.088	62337	6234	-52.930
13.0	0.5	7.72	0.88762	1.37752	23.852	64400	6440	-51.547
16.0	0.5	9.03	0.95569	1.39114	24.611	66451	6645	-50.216
20.0	0.5	10.49	1.02078	1.40416	25.360	68473	6847	-48.944
25.0	0.5	11.70	1.06819	1.41364	25.920	69984	6998	-48.017
32.0	0.5	13.94	1.14426	1.42885	26.844	72480	7248	-46.530
32.0	10.0	13.54	1.13162	1.42632	26.688	72059	7206	-46.777
40.0	10.0	15.06	1.17782	1.43556	27.262	73609	7361	-45.874
50.0	10.0	17.93	1.25358	1.45072	28.230	76222	7622	-44.393
63.0	10.0	18.11	1.25792	1.45158	28.287	76374	7637	-44.308
80.0	10.0	18.29	1.26221	1.45244	28.343	76526	7653	-44.224
100.0	10.0	18.00	1.25527	1.45105	28.252	76281	7628	-44.359
130.0	10.0	17.76	1.24944	1.44989	28.177	76077	7608	-44.473
160.0	10.0	17.96	1.25431	1.45086	28.240	76247	7625	-44.378
200.0	10.0	19.32	1.28601	1.45720	28.655	77369	7737	-43.759
250.0	10.0	18.45	1.26600	1.45320	28.392	76659	7666	-44.150
250.0	25.0	11.48	1.05994	1.41199	25.822	69719	6972	-48.178
320.0	25.0	22.00	1.34242	1.46848	29.409	79405	7941	-42.656

S-33: UNIVERSITY OF NAIROBI SPORTS FIELD (GIANT COURT)

1.6	0.5	29.75	1.4735	1.4947	31.239	84345	8435	-40.093
2.0	0.5	29.05	1.4631	1.4926	31.091	83945	8394	-40.295
2.5	0.5	23.37	1.3687	1.4737	29.767	80370	8037	-42.143
3.2	0.5	16.22	1.2101	1.4420	27.670	74709	7471	-45.244
4.0	0.5	16.02	1.2047	1.4409	27.601	74524	7452	-45.349
5.0	0.5	12.60	1.1004	1.4201	26.307	71029	7103	-47.388
6.3	0.5	5.32	0.7259	1.3452	22.140	59779	5978	-54.708
8.0	0.5	9.58	0.9814	1.3963	24.904	67241	6724	-49.714
10.0	0.5	8.40	0.9243	1.3849	24.258	65497	6550	-50.830
13.0	0.5	7.60	0.8808	1.3762	23.777	64199	6420	-51.680
16.0	0.5	8.03	0.9047	1.3809	24.040	64909	6491	-51.213
20.0	0.5	4.47	0.6503	1.3301	21.383	57733	5773	-56.186
25.0	0.5	7.09	0.8506	1.3701	23.449	63313	6331	-52.270
32.0	0.5	8.74	0.9415	1.3883	24.451	66019	6602	-50.493
32.0	10.0	16.36	1.2138	1.4428	27.718	74838	7484	-45.171
40.0	10.0	17.48	1.2425	1.4485	28.087	75835	7584	-44.608
50.0	10.0	22.04	1.3432	1.4686	29.420	79434	7943	-42.640
63.0	10.0	28.40	1.4533	1.4907	30.950	83566	8357	-40.488
80.0	10.0	38.62	1.5868	1.5174	32.913	88864	8886	-37.878
100.0	10.0	43.76	1.6411	1.5282	33.745	91113	9111	-36.817
130.0	10.0	67.12	1.8269	1.5654	36.760	99251	9925	-33.185
160.0	10.0	64.73	1.8111	1.5622	36.494	98534	9853	-33.493
200.0	10.0	82.96	1.9189	1.5838	38.351	103547	10355	-31.386
250.0	10.0	107.31	2.0306	1.6061	40.376	109016	10902	-29.201

Appendix 8

Foundation conditions and foundation design in Nairobi City Centre

(Courtesy of the Ministry of Roads and Public Works)

Appendix 8 : Foundation conditions and foundation design in Nairobi City Centre

S/No.	Building	Number of floors	Street/ Road	General site conditions	Foundation type	Founding level	Foundation soil	Design Pressures (kN/m²)
1	Continental House		Uhuru Highway	Very weak to 8 m	Pad on 8 by 6m grid with suspended basement slab	4 m	Dark grey tuff	400
2	Sixeighty Hotel	10 with a basement	Kenyatta Avenue	Fairly uniform, weathered to 8 m	Pad	4.5	Black tuff	800
3	Lilian Towers	14 with two split level parkings	University Way	Very soft to 4 m Erratic to 8 m	Pad	7	Agglomeratic tuff	500
4	Sonalux Building	7 with two basements	Moi Avenue	Highly weathered to 11 m, weathered to 20.5 m	Strip	6.5	Brown weathered tuff	150
5	Express Transport Building	6 without basement	Kimathi Street	Fairly uniform, weathered to 6.5 m	Pad	4.2	Black tuff	400
6	Development House II	12 with 2 basements	Moi Avenue	Variable, weathered to 8 m	Pad	6 m	Yellowish brown tuff	400
7	Reinsurance Plaza	20 with two basements	Aga Khan Walk	Weathered up to 9 m	Pad	6 to 7 m	Decomposed rock	
8	Serena Hotel	5 (the proposed was 22 floors)	Kenyatta Avenue	Easterly slope, weathered with clay layer Weathered to 17.5 m	Pads connected with tie beams	6	Tuff	160
9	KICC	28 with two basements	Harambeee Avenue	Fairly uniform, weathered to	Raft for central core		Agglomeratic tuff	

Appendix 8 : Foundation conditions and foundation design in Nairobi City Centre

				15 m	pad for the podium			
S/No.	**Building**	**Number of floors**	**Street/ Road**	**General site conditions**	**Foundation type**	**Founding level**	**Foundation soil**	**Allowable Pressures (kN/m²)**
10	NHC Building	10 with a basement	Harambee Avenue	Fairly uniform, weathered to 10.5 m	Pad	4.5	Grey tuff	500
11	Cooperative Bank building	27	Haile Selassie Avenue	Fairly uniform, weathered to 9.3 m	Pad	4.5	Grey tuff	400
12	Utalii House	12 with two basements	Uhuru Highway	Plastic clay to 5m, weathered to 15 m	Pad	6 to 7	Yellow agglomeratic tuff	300
13	Nyayo House	29	Uhuru Highway	Weathered up to 11 m	Cellular raft	3.0	Black tuff	350
14	Central Bank extension	4 with a basement	Haile Selassie Avenue	Fairly uniform, weathered to 15 m	Pad	4.5	Black tuff	400
15	Chester House	13 with a basement	Kenyatta Avenue	Fairly uniform, weathered to 20.4 m	Pad	6 m	Light brown/ black tuff	600
16	Church House	6 with a basement	Moi Avenue	Variable, weathered to 7.5 m		3 m	Light brown tuff	200
17	Coffee Board	11 with a basement	Haile Selassie Avenue	Fairly uniform, weathered to 15 m	Pad	3.8	Brown/grey tuff	500
18	Ex Telecom Centre	10 with two storey podium	Haile Selassie Avenue	Fairly uniform, weathered to 10 m	Pad-Tower -Podium	3.6 1.5	Yellow tuff Black tuff	400

Appendix 8 : Foundation conditions and foundation design in Nairobi City Centre

S/No.	Building	Number of floors	Street/ Road	General site conditions	Foundation type	Founding level	Foundation soil	Allowable Pressures (kN/m²)
19	Hill Plaza	10 with two storey podium	Ngong' Road	Uniform, weathered to 2 m	Pad	2.4	Grey tuff	400
20	Herani House	?	Kenyatta Avenue	Fairly uniform, weathered to 18 m	Pad	3.0	Brownish black tuff	400
21	International Life House	15 with a basement	City Hall Way	Weathered to 18m soft and fissured to 30m	Pad		Agglomeratic tuff	650
22	City Square Post Office	7 with a basement	Haile Selassie Avenue	Fairly uniform, weathered to 11 m	Pad	4	Brownish grey/Black tuff	500
23	Ardhi House	12 with a basement	Ngong Road	Very erratic	Pad	3.6	Soft decomposed tuff	400
24	Hilton Hotel	18 - Tower 3-Podium with adjoining Basement	City Hall Way	Variable with cotton clay underlying tuff sheet	Circular raft		Agglomeratic tuff	
25	National Bank of Kenya	18 with 5 storey podium	Harambee Avenue	Weathered and variable	Raft for tower pad for podium	5.0 3.5	Grey tuff Yellow tuff	500 200
26	Jogoo House B		Harambee Avenue	Fairly uniform, weathered to 12 m	Pad		Black tuff	400
27	Magereza House	6 with a basement	Bishop's Road Upper Hill	Uniform and sound	Pad	4 m	Trachyte	

Appendix 8 : Foundation conditions and foundation design in Nairobi City Centre

S/No.	Building	Number of floors	Street/ Road	General site conditions	Foundation type	Founding level	Foundation soil	Allowable Pressures (kN/m²)
28	Treasury Building	14 with a basement	Harambee Avenue	Uniform Weathered and fractured	Pad	3.75	Black tuff	400
29	Uchumi House	20 with two basements	Aga Khan Walk	Variable with weakly cemented tuff to 9 m	Pad	6	Black tuff	600
30	Union Towers	12- Office block 14- Service core 7- Executive Tower	Moi Avenue	Weathered to 9.6	Pad	3	Black tuff	250
31	Former USA Embassy	4	Haile Selassie	Weathered tuff, vesicular tuff		5.5 to 6.0	Black tuff	600
32	Comcraft House	7 with two basements	Haile Selassie	Weathered and clayey	Pad		Black tuff	400
33	Corner House	17 with three basements	Mama Ngina	Fractured, diagonal and vertical clay filled cracks	Raft	12	Agglomeratic tuff	400

(Source: Material Testing and Research Department, Ministry of Roads)

Appendix 9

Design pressures for recent low-rise and medium-rise buildings

(Courtesy of City Council of Nairobi)

Appendix 9: Design pressures for recent low-rise and medium-rise buildings

S/No.	Site/Location	Soil type	q_{design} (kN/m²) range	Foundation depth (m)
3	South C	Soft tuff	200-250	D.O.S
4	Loresho	Red soil	150	Minimum 1 m
5	Eastleigh	Hard rock/ Laterite/ Soil	150-450	1.2 m
6	Riverside Drive	Laterite	400	D.O.S.
9	Park Road		150	D.O.S
11	Tassia	Laterite	150	D.O.S
14	Thome Estate	Laterite Clay	200 100	D.O.S
15	Ngecha	Red loam	150-200	Minimum 1.2 m
17	Gigiri	Red loam	200	D.O.S.
21	Jamuhuri Estate	Silty clay	225	D.O.S.
23	Karen	Clay	150	D.O.S. minimum 1.0 m
24	Umoja Estate	Laterite	200-400	D.O.S
25	Industrial Area	Weathered tuff	400	2 m
26	Westlands	Rock Soil	300-400 150-200	D.O.S
27	Kasarani	Laterite Fill	350 150	D.O.S. minimum 1.2 m
28	Runda	weathered rock	150	D.O.S.
29	Kangemi/Dagoretti	Loose murram	150	D.O.S. minimum 1.2 m
30	Dagoretti /Riruta	Tuff Soil	250- 400 100-200	Between 0,9 and 1.2
31	Langata Road	Rock loose Laterite	600 150-200	D.O.S.
32	Denis Pritt Rd	Weathered rock	200	D.O.S.
33	Naivasha Rd	Soil	200	D.O.S.
34	Lavington	Soil	80- 175	D.O.S.
35	Nairobi West	Stiff clay Hard Laterite	200 350	Between 1.2-1.5
36	Garden Estate	Hard Laterite	250	1.2- 4 m

Appendix 9: Design pressures for recent low-rise and medium-rise buildings

42	Baba Dogo Rd	Rock	400	D.O.S
44	Harambee Avenue	Weathered rock	250	
45	Kirinyaga Rd	Weathered rock	200	minimum 1.5 m
46	Westlands	Soil	150- 200	D.O.S. minimum 1.2 m
47	Parklands	Hard Laterite Soil	300-400 100	D.O.S. minimum 1.2 m
48	Ngong Rd	Hard Laterite	200-300	D.O.S
49	Dubois Rd	Hard Laterite	250	D.O.S
50	Upper Hill	Soil Rock	175- 250 400	D.O.S
51	Kenyatta Avenue	Hard rock	350-400	D.O.S
52	Kileleshwa	Hard Laterite	200-250	D.O.S
53	Upper Hill	Rock	250	D.O.S
54	Lower Kabete Road	Red soil	200	1.2-1.7
55	Industrial Area	Weathered rock	300- 400	D.O.S about 1.8
57	Dagoretti/Riruta	Soil	120-150	D.O.S
58	Kilimani	Laterite	200	D.O.S. minimum 1.2 m
59	Ngara Estate	Laterite	300	D.O.S
60	Arwings Kodhek	Soil	200	D.O.S
61	Pangani	Weathered rock	300	D.O.S
62	Buruburu	Laterite	200	D.O.S min 1.2 m
63	Thompson Estate	Red soil	250- 320	D.O.S min 1.2 m
64	Valley Arcade	Weathered rock	300-350	D.O.S
65	Off Jogoo Road	Tuff	400	D.O.S

(Courtesy of City Council of Nairobi)

Appendix 10

Summary of calculated and selected allowable bearing pressures based on test results from some sites

(Courtesy of the Ministry of Roads and Public Works)

Appendix 10: Summary of calculated and selected allowable bearing pressures based on test results from some sites

Site	Lithology	State of the mass	Range of calculated q_{all} (kN/m^2)	q_{all} (kN/m^2) recommended for design
NACECE	Yellowish grey tuff	Severely weathered	545	500
	Dark grey tuff	Severely weathered	682-1363	600
	Yellow agglo. tuff	Slightly weathered	685	600
NSSF Karen	Yellowish grey tuff	Weathered	660-957	600
	Agglomeratic tuff	Slightly weathered	684-1345	600
GSU Ruaraka	Yellowish grey tuff	Weathered	273-955	500
KPC Industrial Area	Grey laterite		650-1193	600
	Brownish grey tuff	Moderately weathered	2386-6629	2000
	Agglomeratic tuff	Slightly weathered	1259-11534	1200
	Agglomeratic tuff	With horizontal & shear frac	5170	3000
Westlands	Grey pumiceous tuff	Slightly weathered	663-1326	800
Mitihani House	Plastic clay	Soft	50-95	Avoid
	Grey tuff	Weathered	50-1050	400
	Grey tuff	Slightly weathered	1237-2044	1200
General motors	Grey tuff	Hard/jointed	572-4620	800
	Nairobi phonolite	With horizontal & shear frac	1237-8813	1200
PTA Plaza	Yellowish grey tuff	Weathered	409-950	600
	Grey trachyte	Weathered or with hairline fractures	1364-3069	1300
	Grey trachyte	Fresh	8899	6000

Appendix 10: Summary of calculated and selected allowable bearing pressures based on test results from some sites

Lillian Towers	Yellow tuff	Severely weathered	289-333	250-300
	Black tuff	Solid	1705-2641	1600
	Yellow tuff	Decomposed	189-289	250-300
	Dark grey tuff	Cemented	268-558	400
	Brown tuff	Weathered	374-790	350
	Agglomeratic tuff	Decomposed	289	250
NYS Ruaraka	Yellowish grey tuff	Weathered	783-1940	700
	Nairobi phonolite	Fresh	5184-10854	3000
Central Bank	Nairobi Phonolite	Weathered to fresh	4467-26537	6000
NCPD Upper Hill	Greyish brown trachyte	Weathered	6353-10972	4000
	Grey trachyte	Fractured to fresh with weak pockets	649-21600	6000
Development House	Yellowish grey tuff	Decomposed	386-421	350
	Yellow agglomeratic tuff	Weathered	302-3645	600
	Nairobi Phonolite	Weathered	14175	6000
Library Ngong' Road	Trachyte	Weathered to fractured	3267-4212	3000
	Trachyte	Hard and compact	6561- 7506	6000
Kenya Re Plaza	Yellowish grey tuff	Weathered	783-2808	600
	Yellow tuff	Decomposed	0-98	Avoid
	Yellow agglomeratic tuff	Weathered	248-964	500
	Yellow agglomeratic tuff	Slightly weathered	1156-4482	800

(Courtesy of the Ministry of Roads and Public Works)