KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY (KNUST)
KUMASI, GHANA

ESTABLISHING A GEODETIC REFERENCE NETWORK IN MONTSERRADO COUNTY – LIBERIA, USING GNSS TECHNOLOGY

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF MASTER Of SCIENCE DEGREE IN GEOMATIC ENGINEERING AT THE DEPARTMENT OF GEOMATIC ENGINEERING OF THE FACULTY OF CIVIL AND GEOMATIC ENGINEERING, COLLEGE OF ENGINEERING

BY

C. SYLVESTER N. BUNDOO

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DECLARATION

I C. Sylvester N. Bundoo, hereby declare that this submission is my own work towards the award of Master of Science degree and that, to the best of my knowledge, it contains neither material previously published by another person nor material which has been accepted for the award of any other degree of any university, except where due acknowledgement has been made in the text.

C. Sylvester N. Bundoo		
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Student Name	Signature	Date

Certified by:

Dr. Ing. Collins Fosu		
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Supervisor's Name	Signature	Date

Certified by:

Rev. John Ayer		
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Head of Dept. Name	Signature	Date

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ABSTRACT

This project created a reliable first order geodetic reference network within Montserrado County. The newly established reference network consists of six (6) primary points, covering an area of approximately 11.693Km^2 with an average separation of about 8Km. The new network monuments are made of reinforced concrete, solidly cemented in the ground with metal caps (brass markers), and coordinated using the static carrier phase differential GPS measurement. Several observation sessions were conducted and processed to compute 3D coordinates for the network. Post processed data was rigorously adjusted using Least Squares and found to have met the distance accuracy specifications for a first order network (1:100,000). Final coordinates are left in the WGS84 frame to be transformed to Liberia's local grid when parameters are available, and are within a mean accuracy of ±0.05m.

This master thesis is a description of how this geodetic reference network was established; giving detailed explanation of the individual activities that culminated into the finished coordinates for the network.

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ABBREVIATIONS

CORS	Continuously Operating Reference Station
CNDRA	Center for National Documents and Records/Archives
DGPS	Differential Global Positioning System
DLSC	Department of Lands, Surveys and Cartography
GIS	Geographic information System
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
LCS	Liberian Cartographic Service
LOP	Lines of Position
LPIS	Land Policy and Institutional Support Project
MCC	Millennium Challenge Corporation
MLME	Ministry of Lands, Mines and Energy
PDOP	Positional Dilution of Precision
PPM	Parts Per Million
PPP	Precise Point Positioning
TDP	Time Dependent Positioning
USAID	United States Agency for International Development
WGS	World Geodetic System

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1.0 CHAPTER ONE – INTRODUCTION

1.1 BACKGROUND

After nearly two decades of devastating civil conflict, the West African nation of Liberia is making endless strides in developing a suitable land reform and management policy that will contribute to lasting peace and the rebuilding of the Country's post-war economy. The long years of war quickly crumbled the nation's infrastructure and human capital. An estimated two hundred fifty thousand (250,000) people died, more than a million others scattered across the sub region as refugees (Paczynska, 2010), and the Nation's geodetic infrastructure, like all others, almost completely destroyed. Key agencies of government concerned with land, such as the Departments of Lands, Survey and Cartography (DLSC) and the Center for National Documents and Records/Archives (CNDRA), were dilapidated by the years of conflict and neglect. This infrastructure breakdown now serves as a potential catalyst for small and large scale land disputes in post-war Liberia.

According to a recent survey, violent conflicts in Liberia have their root in land rights issues 63% of the time, significantly more than political, ethnic, or religious differences (Paczynska, 2010). In its 2008 final report, the Liberia Truth and Reconciliation Commission (LTRC) acknowledged the threat pose by land disputes to national peace and the likelihood of a return to war if the issue is not addressed (LTRC, 2008). Individual and small scale land conflicts in Liberia have erupted for a variety of reasons, but most frequently due to poorly demarcated and documented land rights.

Most properties in Liberia lack spatial definition. This lack of spatial data contributes to a number of land disputes as property rights, guaranteed by deeds, have limited value if they do not contain details as to the physical location of the actual property. Without this spatial information, the only reference to a location in the deed books is a metes and bounds description of the location of the property, or a lot number which is applicable in urban areas. The metes and bounds description is not suitable to Liberia where the topography and vegetation change rapidly (MCC, 2011).

However, properties cannot easily be defined spatially and their physical location cannot be easily ascertained in the absence of a Geodetic Reference Network that is accessible and reliable, to which property surveys must be tied. Unfortunately, most of the monuments and benchmarks defining Liberia's only existing network have been destroyed by the long years of war. There is also little or no information about the few that survived the War, thus creating a continuous drag on numerous efforts by government and foreign partners aimed at improving the land administration sector. There is therefore a compelling need for the establishment of a first order geodetic network that connects to the existing network at a high degree of accuracy.

1.2 Problem Statement

The one and a half decades of war almost completely destroyed Liberia's geodetic infrastructure; dilapidating its physical component and completely draining its human capital. The prevailing system of compass-survey and/or metes and bounds description of properties in Liberia is fast becoming unattractive to property owners given its limitations in spatially defining property boundaries. Most land disputes in Liberia erupt as a result of poorly demarcated boundaries that are not retraceable in the real world. A reliable coordinate reference system is a basic requirement for the successful execution of all survey related projects. The definition and densification of coordinate reference systems hinge on points whose 2D or 3D positions are known to a high degree of accuracy (Acheampong, A. A., 2008). Sadly, most of the points defining the existing network in Liberia were damaged during the long years of war, and there is little or no information concerning the surviving ones.

Since the end of the Civil war, numerous efforts have been made by the Ellen Sirleaf led-government aimed at salvaging the land administration sector in general and the geodetic infrastructure in particular. In August 2009, the Liberia Lands Commission was formed by an Act of Legislature (LLCA, 2008) as an autonomous government body. The purpose of the Commission is to propose reforms to land policies and laws as well as to coordinate the various land activities underway in the country. Currently, plans are underway to establish an accessible first order geodetic network in Liberia in keeping with recommendations of the Millennium Challenge Corporation (MCC) to the DLSC consistent with component one (1) of the US $15 million threshold grant agreement signed by MCC and the Liberian Government in July, 2010. (MCC, 2011)

Both the existing and the proposed networks consist of first order points spread throughout the Country at very large intervals, sometimes inaccessible by surveyors in some parts of the country. Moreover, survey measurements connected to very far away points are prone to errors and inaccuracies. There is therefore a compelling need for the establishment of a first order geodetic network that connects to the existing network at a high accuracy and shorter intervals.

This project aims at creating a reliable, accurate reference network, connected with the existing local frame that will be accessible by everyone without any charges. It will also serve as a pilot project for the future densification of the National Geodetic Network.

1.3 Project Benefits

In order to permit the many and varied surveying, mapping and charting programs to be referenced to some common system, it is necessary to have a common reference framework of control points. An accurate framework can be useful for various levels of users. A reliable coordinate reference system is a basic requirement for the successful execution of all survey related projects. The definition and densification of coordinate reference systems hinge on points whose 2D or 3D positions are known to a high degree of accuracy (Acheampong, 2008).

For accurate and efficient policy decisions to be taken, the development and utilization of Geographic Information System whose accuracy and effectiveness is dependent on a suitable Geodetic Control Network cannot be overemphasized (Arinola, 2006). The development of Satellite technology, especially its application in geodesy through the use of GPS has opened a new vista in the observation and strengthening of Geodetic Control Networks worldwide (Arinola, 2006). It is therefore my ardent conviction that this project will immensely benefit Liberia in the fields of surveying and GIS-dependent policy decisions.

1.4 Project Objectives

The main aim of this project was to create a network of survey control points in Monrovia.

The specific objectives of this project were to:

- Provide coordinates in the GPS System for the Network;
- Ensure the safety and security of the network by recommending necessary protective measures to be implemented by DLSC; and
- Provide description cards for each point in the network with sufficient information for easy access.

1.5 Research Questions

The following questions were asked and addressed:

- What is the status of the existing network?
- What are the problems caused by the lack of a reference network?
- What are the accuracy requirements of the new network?
- What measures are taken to protect and preserve the network?

1.6 Scope of Project

This project was undertaken to establish a new geodetic reference network in Liberia. The new network was established in Monrovia, Montserrado County, Liberia, and consisted of six (6) first order control stations made of pre-cast reinforced concrete beacons. The project was implemented over a period of six (6) months commencing the first week of March, 2013. This network is established to take advantage of the latest GPS technology in the field of surveying to minimize the numerous problems resulting from the inaccuracies caused by compass surveying methods.

As stated above, completion of the project must be achieved by August 2013. Progress milestones associated with the project are as follows:

- Complete field reconnaissance report by March 22, 2013;
- Design plan (map) of new network by March 23, 2013;
- Approved project permit from MLME on March 24, 2013;
- Monumentation, from March 26, 2013 to March 31, 2013;
- Field measurements, from April 7, 2013 to April 12, 2013;
- Data processing and analysis, from April 13, 2013 to May 25, 2013;
- Complete project report by August 5, 2013; and
- Project presentation (final defense) on August 23, 2013.

1.7 Study Area

The project area is Monrovia, the capital city of Liberia and a district in Montserrado County, is located between Latitudes 6°15'00"N and 6°30'00"N and Longitudes 10°30'00"W and 10°45'00"W. It is bounded on the North by the St. Paul River, on the East by Margibi County, on the West by Cape Mesurado and on the South by the Atlantic Ocean. See map below.

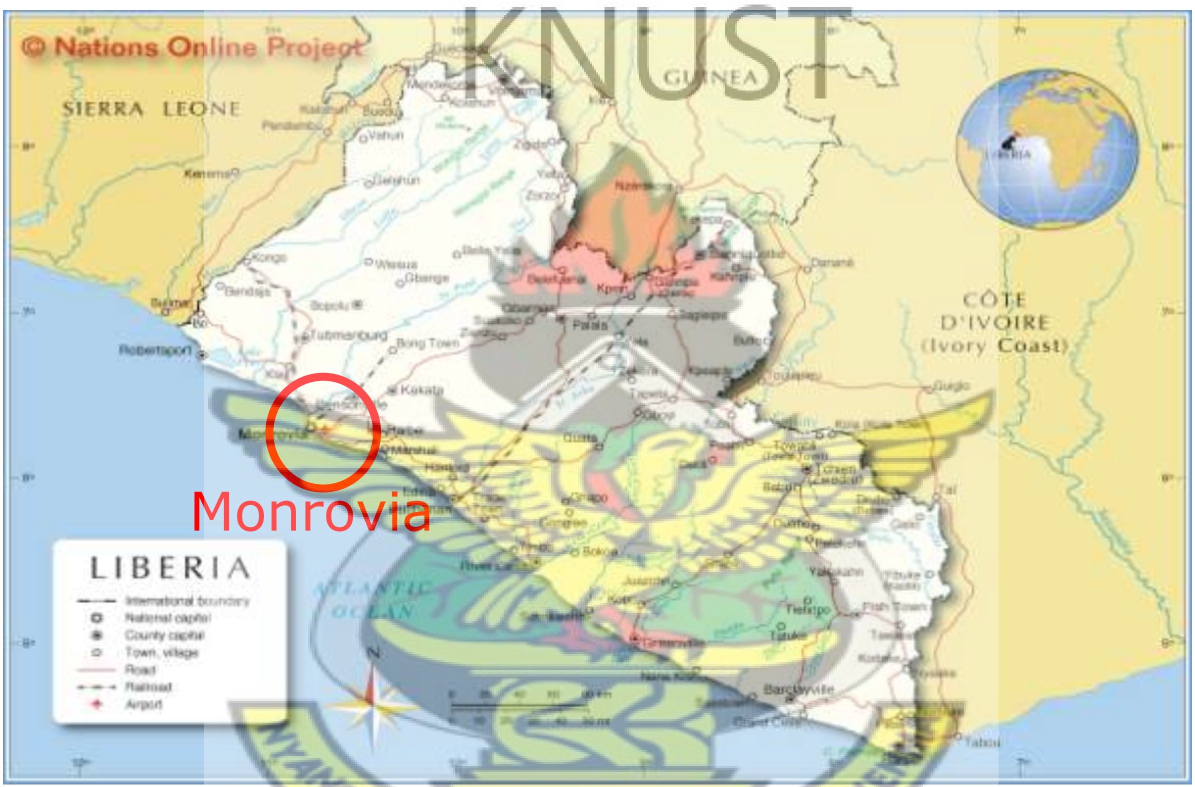

Figure 1: Map of Liberia showing the location of the study area (Monrovia)

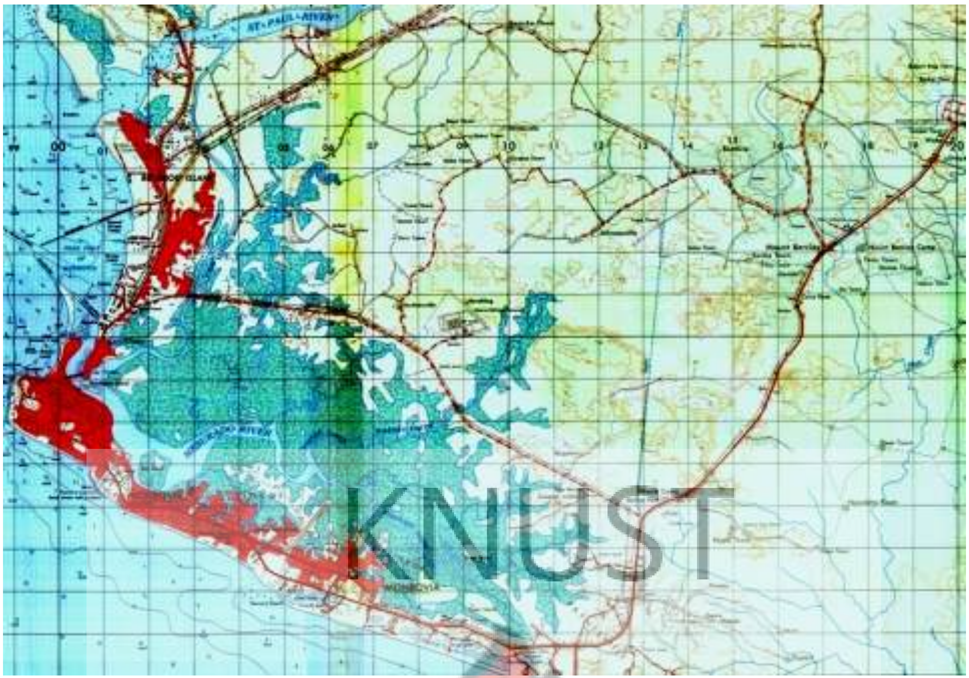

Figure 2 : Project Area

1.8 Thesis Outline

This report is organized in six chapters. Chapter One introduces the entire project; the background, the problem statement, project objectives, benefits, project scope, and study area. Chapter Two provides an overview of geodetic control networks; their types, design, accuracy classification, methods of establishment, and essence.

Chapter Three takes a look at the methodology used for the project; the pre-field activities, the field work, and the post-field work. Chapter Four takes a look at the data processing procedures; pre and post-processing techniques, results obtained and adjustments to arrive at final coordinates. Chapter Five delves into the results and discussion; the interpretation of results obtained and what they mean. Chapter Six gives a conclusion on the entire work and ends by giving some recommendations for future works.

2.0 CHAPTER TWO – REFERENCE NETWORKS

Before venturing into a practical project, a thorough theoretical knowledge of the task is required. To establish a geodetic network, one must have an understanding of the various types of geodetic networks, their design, accuracy requirements, and essence. This chapter gives a brief overview of geodetic control networks; their types, design, accuracy requirements, and importance.

A geodetic reference network is the wire-frame or the skeleton on which continuous and consistent mapping, Geographic Information Systems (GIS), and surveys are based (I-Plan, 2003). Traditionally, geodetic control points are established as permanent physical monuments placed in the ground and precisely marked, located, and documented. Locating spatial features with respect to geodetic control enables the accuracy assessment of these features (I-Plan, 2003). Interest and activity regarding geodetic control has dramatically increased at all government levels because of the need for accurate maps and surveys needed for national development and environmental studies.

2.1 Types of Geodetic Control

There are three basic types of geodetic control: Horizontal, Vertical and Gravity. This thesis will focus on horizontal and vertical controls. Gravity networks will therefore not be discussed in this document.

2.1.1 Horizontal Control

In surveying, a horizontal control is a system of points whose horizontal positions and interrelationships have been accurately determined for use as fixed references in positioning and correlating map features (Sci-Tech-Dictionary). Horizontal control can be established by a number of methods, the commonest being Triangulation, Trilateration, and Precise traversing.

Using triangulation, the locations of survey points are calculated using the measurement of angles in a network of triangles. Lines of sight are obtained by placing the stations on high points. Angles are then measured to other distant points and networks of connected triangles are formed. A similar method of creating networks of connected triangles is Trilateration. Distances between stations, rather than angles, are measured.

It is assumed that a precise traverse method would be used where triangulation would be impracticable, that an absolute accuracy of 1 in 100,000 would be required, and that reasonable speed would be necessary (Clappison, N. O, 1962). In this method the spots of the points in a given coordinate system are resolved using dimensions of the lengths of the lines that successively connect them. Horizontal angles between the lines are measured using Theodolites and Transits. The objects of observation are usually special marks set up and they are mounted at the stations being sighted. Steel or invar tapes or wires are used to measure the length of traverse legs and nets. Suitable corrections are made in the length and angle measurements so that coordinates with higher accuracy are obtained. It is required that the locations of the traverse stations should be stanched[1].

The fourth method of establishing horizontal control is GPS, which avoids line-of-sight problems and is faster, cheaper, and more accurate than traditional surveying methods.

Horizontal geodetic control data consist of distances, directions, and angles between control stations. This data is used to determine geodetic coordinates and azimuths. The geodetic

[1] http://classof1.com/homework_answers/civil_engineering/geodetic_surveying/

coordinates (Latitude and Longitude) can be converted to other coordinate systems by applying basic principles in Geodesy.

2.1.2 Vertical Control

Vertical control networks are a series of points on which precise heights, or elevations, have been established. Vertical control points are typically spaced in close proximity to one another, typically along railroads or highways (ASCO, 2005). Vertical control stations are typically called bench marks. As a part of a vertical information network, the bench mark's elevation is known relative to a datum, usually mean sea level. Normally, the orthometric height is known. The orthometric height is defined as the distance between the vertical reference surface (geoid) and the vertical geodetic station on the surface of the Earth measured along the plumb line between the two. Differential leveling is the commonest method of determining elevation (ASCO, 2005).

In differential leveling, a sequence of lines-of-sight is established. Two readings are taken along the line of sight, one at a known benchmark elevation and the other at a point of unknown elevation. The difference between the two is used to establish the elevation of the unknown point. For geodetic work, three-wire leveling is used (ASCO, 2005). In this method, a leveling instrument with an eyepiece having three separate horizontal lines is used. Elevation is determined by finding the average reading for each of the horizontal lines. GPS can also be used to obtain vertical heights. However, GPS only gives accurate ellipsoidal heights.

2.2 NETWORK DESIGN

Network design, for the purpose of this document, includes the determination of the number and location of existing control stations for network constraints, the selection of new project station locations, and the relative dispersion of network observations.

Space-based measurement systems, such as GPS, are not significantly affected by such factors as network shape or intervisibility (JPO, 1991; Leick, A, 1992). This provides the opportunity to focus upon the intent of the project control, and geographically sensitive issues rather than the limitations of the measurement technique. However, network design does have relevance both for the elimination or reduction of potential error sources as well as for providing adequate ties to the existing geodetic reference system. These concerns may be addressed by the choice of which existing control stations should be included, as well as the planning of new station locations and network observation periods (CGCC, 1996).

To meet a local network accuracy classification, a GPS project must be connected to sufficiently accurate and well distributed existing control (CGCC, 1996). All of the control stations to which the network will be constrained must have positions known on the same datum and epoch since additional uncertainty in the time-dependent positioning models (TDP), which would need to be added into the network constraints, will likely add unacceptable levels of uncertainty to the network (Snaye, R. A., 1993).

2.3 Network Accuracy Classification

According to the Federal Geodetic Control Subcommittee of the United States, when points are established in any particular survey, they will have datum values consistent with other points of the same classification. Classifications are not determined by observation closures within a survey, but by the ability of the new surveys to duplicate already established values (FGCC, 1984). See Tables 1&2 for a listing of the classifications for order and class of accuracy.

Table 1: Distance Accuracy Standards[2]

CLASSIFICATION	MINIMUM DISTANCE ACCURACY
A-Order	1:10,000,000
B-Order	1:1,000,000
First-Order	1:100,000
Second-Order, Class I	1:50,000
Second-Order, Class II	1:20,000
Third-Order, Class I	1:10,000
Third-Order, Class II	1:5,000

The order and class that is stated for any point or set of points provides a relation of specific accuracy to other points in the survey. This relationship is expressed as a distance accuracy. The distance accuracy is the ratio of the relative positional error of a pair of control points to the horizontal separation of those points.

[2] Standards and Specifications for Geodetic Control Networks (1984)

Table 2 : Elevation Accuracy classification[3]

CLASSIFICATION	MAXIMUM ELEVATION DIFFERENCE FOR LOOP OR LINE	
	Metric	English
First-Order, Class I	$4mm\sqrt{K}$	$0.017\sqrt{M}$
First-Order, Class II	$5mm\sqrt{K}$	$0.021\sqrt{M}$
Second-Order, Class I	$6mm\sqrt{K}$	$0.025\sqrt{M}$
Second-Order, Class II	$8mm\sqrt{K}$	$0.033\sqrt{M}$
Third-Order	$12mm\sqrt{K}$	$0.050\sqrt{M}$

K equals the distance in kilometers and M equals the distance in miles. When expressing an order and class of accuracy as relating to vertical control points, the relation is expressed as an elevation difference. Elevation difference accuracy is the relative elevation error between a pair of control points that is scaled by the square root of their horizontal separation traced along existing level routes.

2.4 Transformations

A DATUM is a point, a line, or surface used as a reference in surveying and mapping. A geodetic datum is a mathematical model of the earth used to calculate the coordinates on any map. Many countries use their own datum when making their maps and doing their surveys. These are referred to as local datums. The same area may have two datums giving each point two different coordinates. In addition datums generated by individual countries do not match at their boundaries (NGA, 2013). Transformation equations and parameters provide a means

[3] Standards and Specifications for Geodetic Control Networks (1984)

of transforming coordinates referenced to one datum into coordinates referenced to a different datum.

A seven parameter (three dimensional) transformation can be used to relate Cartesian Coordinates expressed in two different systems.

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_B = \begin{bmatrix} \Delta X_0 \\ \Delta Y_0 \\ \Delta Z_0 \end{bmatrix} + S.R. \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_A \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad 2.1$$

$(\Delta X_0, \Delta Y_0, \Delta Z_0)$ are the translation parameters, S is the scale factor, R contains the rotation parameters, and $(X, Y, Z)_A$ and $(X, Y, Z)_B$ contains the coordinates in the two systems.

2.5 Network Adjustment

It is an established fact that no measurement can be made to perfection. Due to human limitations, imperfect instruments, environmental effects and improper measurement routines, which together define the measurement condition, all measurement results most likely contain errors. To account for these errors and properly integrate the new points into the old geodetic system with higher accuracy, a network adjustment has to be done. Network adjustment helps to minimize the errors in the measurement. In this project, a computational process known as least squares (weighted least squares) was used for network adjustment.

3.0 CHAPTER THREE – METHODOLOGY

Conventional methods for establishing horizontal and vertical controls (traversing, triangulation and/or trilateration, for example) are expensive, tedious and limited to intervisibility between beacons and the area of survey, Satellite Positioning Technologies have been proven to be fast, simple, accurate and cheaper alternative to conventional methods (USACE, 1996; Fosu, C., 2001). Tracking signals from Global Navigation Satellite Systems (GNSS) will help in achieving higher accuracies and overcoming numerous limitations associated with terrestrial methods. The method adopted for this project is therefore that of satellite positioning technology (DGPS observations).

This chapter outlines the activities and materials used for the execution of this project. These activities are discussed in the order shown below:

- Materials used
- Desk Study
- Project Planning
- Field Reconnaissance
- Monumentation
- Field Measurements
- Point Description
- Point Names

3.1 Materials used

The following were used for the project;

- A topographic map, 1:50,000, of the project area and one Gamin hand-held GPS were provided by the DLSC for field reconnaissance.

- One set of Leica DGPS receivers (one base and two rovers) was used for field measurements/observations.
- One digital camera and field data entry sheets were used
- Six (6) 1st order reinforced concrete monuments were constructed as per the specification provided by the DLSC.
- MatLab was used for adjustment computations.
- Leica Geo-office was used for the initial post-processing in Monrovia.
- Leica spiderQC and Sokkia Spectrum Survey were used in Ghana for final post-processing.

3.2 Desk Study

Creating a new geodetic reference network requires a good knowledge of existing points within the project area with both local and WGS84 coordinates. This is necessary for the purpose of relating the new points to the old network in the best way possible. It is against this background that relevant state authorities were contacted for available information on the existing network.

Authorities at the DLSC were contacted and they provided relevant information on the existing network. DLSC also provided a 1:50,000 topographic map of the project area on which six (6) 1st order controls within the study area were identified. The actual status of the six (6) points identified was unknown and it was uncertain if they could be used for the project. One major reason for the uncertainty of using some of the points for this project was the extreme heights at which they were placed. They were no longer accessible during the time of this project and GPS readings were not possible at their locations. Another reason was the lack of original data in both the local Liberian system and the WGS84 system for the available points. Authorities at the DLSC clearly stated that the long years of war destroyed all available information on the geodetic infrastructure. The geodetic office at the DLSC however provided UTM and Geographic Coordinates of Camp Ramrod and few other stations. Regrettably, the office fell short of clarifying how these coordinates were derived.

During this period of pre-field work, a series of meetings were held with both requisite government agencies and supervisors at KNUST. These meetings and exchanges paved the way and set the agenda for the successful execution of this project.

3.3 Project Planning

Planning for this project was done in two phases; phase one involved activities covering field reconnaissance to monumentation. Phase two included activities ranging from field measurements to post-processing.

Phase one focused on planning the sequence of visitation to existing points considering the available resources. Plans were also made for the locations where the new monuments were to be constructed, their curing period, and the mode and order of transporting them to their required destinations. It is worth noting here that transportation of the new monuments required careful planning as much energy and man-power were needed to move precast reinforced concrete beacons to various communities. There was an incidence where the weight of the beacons caused the car to fail on a hill, resulting into an accident that nearly caused a major setback.

Figure 3 : Jeep carrying survey beacons being pulled after the accident

Phase two was basically session planning. For the purpose of collecting accurate data, consideration was taken on periods of the day when there will be at least four satellites in view and a suitable corresponding Positional Dilution of Precision (PDOP). Session planning for this project considered the following steps, as provided in the Minnesota SM_Manual.

- A plot was made, displaying all the control stations that were to be occupied and the planned vectors between them.

- Since only one session was going to be observed per day, sessions were numbered using the dates on which they were observed. Eg 09/04/13 meaning this session was observed on the 9th of April, 2013.

- Lengths of sessions were determined based on the results obtained by the following formula, which sets 30 minutes as the minimum for a static GPS survey:

$$L = \frac{max.baseline\ (m)}{125}\ X\ \frac{4}{\#\ of\ satellites\ in\ view} \qquad \text{. . . . . 3.1}$$

L is the length of session in minutes. However, for the purposes of obtaining higher accuracies and using the method of PPP when necessary, all sessions were observed for a minimum of three (3) hours. This time exceeded all session lengths computed from the above formula.

- Logistics were properly planned, such as time needed for moves between stations, matching the right operators for the more difficult assignments, and the efficient use of "Leapfrogging" techniques. finally,

- Field session forms, station location maps, descriptions and work assignments were prepared for completion during the sessions.

3.4 Field Reconnaissance

Reconnaissance for this project included two major activities; inventory of existing network points, and identification of sites for the new network stations. This activity ended with a brief report on the existing network and a conceptual design of the new network.

3.4.1 Inventory of Existing Points

Earlier review of the topographic map of Monrovia showed that there were six (6) primary control points within the study area. However, field trips to those points showed that three (3) were completely damaged by the long years of war and neglect, while one of the remaining three (3) was partially in existence but inaccessible. Most of the points in the old network were established using conventional triangulation methods. The points are therefore situated at extreme heights that are currently inaccessible.

Current photo of JFK Hospital   Pre-war photo of Ducor Hotel

Figure4 : Two Existing controls on top of JFK Hospital and Ducor Hotel

Of the remaining two (2) stations within the study area, only one was readily accessible during this project period. The point at JFK, in the figure above, was very difficult to access due to administrative bureaucracies at the Hospital. It was therefore decided that the only available station, Camp Ramrod, be used as the base station due to its ease of access.

Figure 5 : Camp Ramrod; Existing station

It was also decided that Camp Ramrod be fixed by the method of Precise Point Positioning (PPP) since data provided by DLSC could not be relied upon. This was decided to have at least one fixed point in the network. This may not have been the ideal solution, but the best given the duration of this project and the circumstances under which it was executed.

3.4.2 Sites Identification for New Stations

The stability and survival of any survey marker is heavily dependent on its location, and the resultant accuracy of any survey is partly dependent on the stability of the mark. With this knowledge, station sites for this project were selected based on the following site selection parameters, as provided by the US Army Corps of Engineers Manual (USACE, 2012).

3.4.2.1 Security

Foremost on the list of evaluation considerations is the mark's susceptibility to damage or destruction. In view of the great expense involved in establishing a mark and the data associated with it, time spent in preservation is worthwhile. Sites for this project were selected with due anticipation of any construction, future highway, waterway, or any work that might occur at the proposed mark location. Public facilities were therefore selected for all the new stations to increase their security.

3.4.2.2 Accessibility

Accessibility of the marks by users was evaluated in selecting the marks' sites. All selected sites have permanent nearby structures or objects to which they are referenced. Distances and directions, establishing lines of position (LOP), are given from those structures to enable easy access. Additionally, locations with good topography were selected to enable convenient occupation of points with various survey equipment.

3.4.2.3 Stability

All marks are subject to the effects of geological and soil activity. Liberia's equatorial position on the globe excludes it from the list of countries that experience more severe environmental effects that adversely affect the stability of survey points. Locations with stable soils, mixture of clay and gravel, or rocky areas were selected to ensure the stability of the points.

3.4.2.4 Satellite Visibility

In addition to permanence, security, utility, and stability, satellite visibility was also considered during sites selection. Locations for the new stations were selected with due consideration to technical limitations of GPS. Points are situated in locations with a clear view of the sky above 15^0, far from radio transmission towers, and far from flat surfaces and water bodies that could reflect signals and increase multipath errors.

3.4.3 Existing Geodetic Network

Little is known about the existing geodetic network in Liberia. The long years of conflict and neglect paralyzed the entire geodetic infrastructure. The Geodetic Department of the LCS was only able to provide information on four (4) first order stations within Montserrado County, which are part of the existing network (Table - 1 below).

Table 3 : Coordinates of Existing Geodetic Points provided by DLSC

Stn. ID	UTM Zone 29 Coordinates		Geographic Coordinates		Elevation (m)	Stn. Location	Remark
	Eastings (m)	Nothings (m)	Latitude	Longitude			
TP03[E]	302088.880	701044.360	6°20' 23.2876" N	10°47' 20.8671" W	----	CEMENCO, Monrovia	Inaccessible
TP01[F]	303752.140	695158.780	6°17' 11.8767" N	10°46' 26.1031" W	39.2100	JFK Hospital Monrovia	Difficult to access
RAMR[B]	311630.840	696206.820	6°17' 46.8480" N	10°42' 09.9120" W	16.0949	72nd Barracks Monrovia	Accessible
ITT2[A]	299432.510	698864.300	6°19' 12.0200" N	10°48' 47.0400" W	105.4670	Ducor Hotel Monrovia	Inaccessible

According to DLSC, most stations of the existing network are points of the first order traverse that run across the country from the Roberts Field International Airport northeast to a point near the Guinea border in the Nimba Mountains.

These points were established by observations made in 1964-1965 by the 72nd United States Army Engineer Survey Liaison Detachment in the frame of the Joint Liberia - United States Mapping Project. First order astronomic position observations were performed in several parts of Liberia (the Nimba Mountains, Zwedru, Foya, Suakoko, etc). The network is known as the Liberia 1964 Datum with fundamental point at Roberts Field Astro_Latitude = 6° 13' 53.02" ± 0.07" N, Longitude = 10° 21' 35.44" ± 0.08" W, Elevation = 8.2331 m, and Azimuth = 195° 10' 10.57" ± 0.14" to Roberts Field Astro Azimuth Mark from south, on the Clarke 1880 ellipsoid (MCC, 2011). The First order stations are monumented by concrete pillars with forced centering devices.

(a) Side view (b) Plan view

Figure 6 : First Order Station (RAMROD ASTRO[6])

3.4.4 New Network

The new Network is a 3D control, i.e. every control point has horizontal coordinates (longitude, latitude and plane coordinates) and ellipsoidal height. The Network consists of Six (6) primary GPS stations spread across Monrovia and its environs. The monument locations were planned in collaboration with the DLSC.

The new GPS stations were installed on public facilities to (1) insure the safety of the points and their easy accessibility, and (2) lay the foundation for their future development into CORS stations.

The new GPS stations were monumented by reinforced concrete beacons solidly cemented in the ground with survey makers, made of brass, inserted at the center of each monument. (Figure 7).

Figure 7: New Station Monument

3.5 Monumentation

Monumentation for this project comprised two major activities; monuments construction, and monuments installation.

3.5.1 Monuments Construction

The new GPS stations were constructed with pre-cast reinforced concrete. The monument specification was provided by the DLSC, while Assistant Minister Miller provided the monument markers. The markers, made of brass, had the inscription:”Liberia Ministry of Lands, Mines, and Energy; Survey Marker; Do not disturb”.

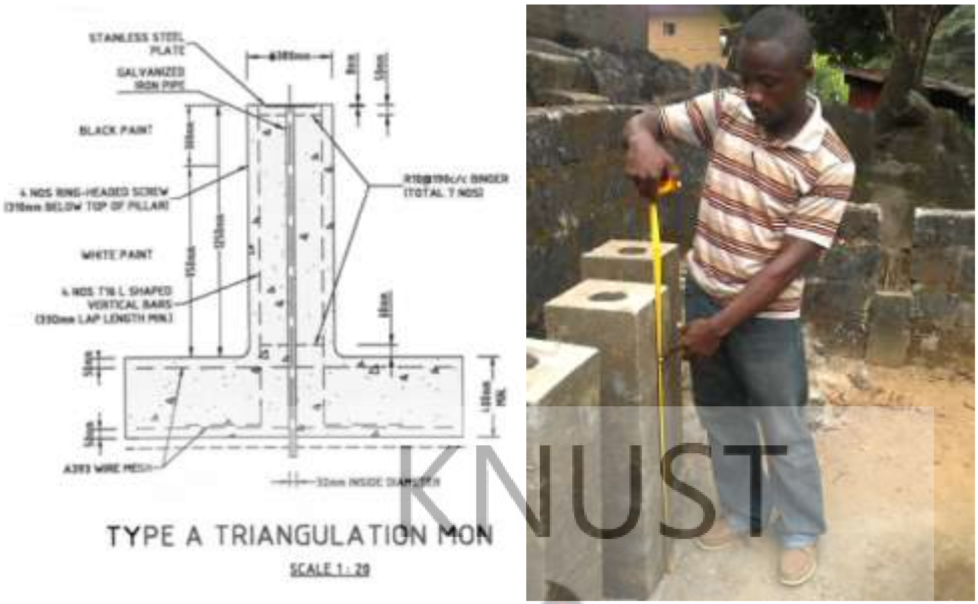

Figure 8 : Monument spec. (L), and Constructed Monuments (R)

3.5.2 Monuments Installation

The constructed reinforced concrete beacons were solidly cemented in the ground in strict compliance with the specifications and guidelines given by the DLSC. A (3 X 3 X 3)ft excavation was made in the ground at each station site. A foundation of rocks and concrete was laid to a height of one (1) foot. The monument was then inserted and made to rest on that foundation with 2ft of the monument buried in the ground. A mixture of rocks and concrete was then pulled at the sides of the monument once it was centered and held vertically upright, up to ground level to ensure its stability. Survey makers, made of brass, were then inserted at the center of each monument, followed by finishing works and labeling.

(a) Installing a control monument (a) An installed & finished control monument

Figure 9 : Monuments installation

The verticality of the points was checked both before and after the finishing works by placing a masonry level at the sides and top of the monument and making sure the water bubble is correctly centered.

3.5.3 Point Names

Given the limited time for this project and Mr. Bismarck Harding (Chief geodetic surveyor) out of the country, information on DLSC naming system could not be provided. A temporary naming convention for the new network was therefore created. Since the new stations are monumented with concrete beacons, it is easy to rename the points any time to conform to DLSC naming standard.

Each point name consists of two parts, a prefix of two letters and a number (GP1), and a simple serial number followed by the name of the place. The GP1 represents first order geodetic point. For example; ***GP1-2 Freeport*** means the second first order geodetic point of the network located at Freeport.

3.6 Field Measurements

Field measurements were carried out over a period of four (4) days; April 09 to April 12, 2013. The long distances between the points, the usual traffic jam in Monrovia, and having only one vehicle did not permit us to have more than one session per day. We were divided into two teams; one headed by Mahmoud and the other headed by me. We met every morning to review the daily scheduled and met again after each day's work to make sure everything worked as planned. During each day's work, we remained in constant contact with each other and our supervisors at KNUST via mobile phones to make sure the right thing was done.

The static carrier phase differential measurement was used for the observations. Three (3)-hour sessions were observed at each point, and three receivers were used simultaneously (one base and two rovers) during each observation. This method forms a triangulated network of baselines and gives the network a good stability. It also simplifies the calculations and makes error detection easier in the formed baseline loops (Lantmäteriverket, 1996).

A separate ten (10)-hour observation was carried out at the base station to fix it using Precise Point Positioning method as data given for it was not reliable. The GPS observation sessions involving rover stations were carried out using suitable field procedures given in (FGCC, 1998) and (USACE, 1996). Antenna heights for both the base and rover stations were carefully measured and recorded before and after each session.

Field measurements were also done around each point, detailing permanent nearby structures and road intersections, to give a graphic description of the points for the purpose of easily locating them.

The Base station at 72nd (Camp Ramrod)

A Rover station at Omega Tower

Figure 10 : An observation session

3.7 Data Processing

Data processing for this project underwent two stages: preliminary and final processing. Preliminary processing was done in Liberia immediately after data collection. Field data was initially processed using Leica Geo-office software. The baselines were computed holding Camp Ramrod at 72nd Community fixed. This preliminary processing was carried out to judge if the data collected was of adequate quality for this project.

Final processing for this project took place in Ghana using the Sokkia Spectrum Survey Software. The base station was fixed by the method of Precise Point Positioning (PPP) with the aid of an online processing service in Canada. Precise ephemeris data was used for the purpose of achieving higher accuracy.

An improved set of positions were obtained using least-squares minimization procedures and equations modeling potential error sources. A detailed explanation of the data processing procedure can be found in Chapter Four.

3.8 Testing of Processed Results

Post-processed results were tested to analyze the data for internal consistency and to eliminate possible blunders before applying full least squares adjustment. A unit variance test was also conducted to check the proximity to unity of the unit variance.

3.9 Results and Analysis

Final results obtained from the Least Squares Estimates are presented in tables 11 & 12 in Chapter Five. Each table is preceded or followed by analysis of the results it presents.

3.10 Point Description

As stated earlier, measurements were done around each point, detailing permanent nearby structures and road intersections. This was done to give a graphic description of the points for the purpose of easily locating them. Distances and directions, establishing lines of position (LOP), are given from those structures. Additionally, a separate description card (as presented in Appendix A) is prepared for each point. These point description cards contain;

- The point's name
- The point's location
- The type of monument
- The date of establishment
- The establishers
- UTM and Cartesian Coordinates of the point
- Satellite image of the area
- Ground photo of the area showing the point, and
- A short narrative description of how to get to the point.

4.0 CHAPTER FOUR - DATA PROCESSING

It is important to process GPS data immediately after data collection to identify problems while it is easy to remedy them. Data processing commenced as soon as data and field log sheets were returned from the first day of observations. Data submissions were verified to ensure they were clear, complete and accurate. After these, data was then downloaded from the receiver or its data storage media to the computer. Backups were made of all raw data. (GPS-Guide, 1993).

This chapter explains the sequence of activities involved with the processing and adjustment of coordinates that culminated into the final coordinates for the new network stations. Below is the order in which processing and adjustment procedures are discussed:

- Preliminary Processing
- Final processing
- Network Adjustment

4.1 Preliminary Processing

Preliminary processing was done in Liberia immediately after data collection. Field data was pre-processed on the 13th of April at LPIS work station in Sinkor-Monrovia, using Leica Geo-office software. The baselines were computed holding Camp Ramrod at 72nd Community fixed. This pre-processing was carried out to judge if the data collected was of adequate quality for this project. Pre-processed results were analyzed to assess the goodness of the data, compare redundant measurements, and make decisions as to the need for any re-observations or revisions of plans.

4.2 Final Processing

The complexity of data processing corresponds with the complexity of the GPS technique used. Processing for conventional static GPS surveys is more complex and may require combining several sessions of observations. All data for one session must be loaded onto a computer. The appropriate "known" three-dimensional coordinates of the control points (fixed Points) should be entered in the processing program as well.

The data for the base station was converted to RINEX format and sent to the online processing station in Canada for Precise Point Positioning (PPP). As earlier stated, there was a need to have at least one fixed point in the Network since there were no known points.

Since this project will be used for cadastral work, a higher accuracy is required. To achieve this, precise ephemeris data was downloaded from the internet, and the rest of the data converted to RINEX format and processed with Sokkia spectrum Survey.

Office reduction procedures were conducted to generate GPS baseline solutions through an iterative process, and observed values were compared to computed values. An improved set of positions were obtained using least-squares minimization procedures and equations modeling potential error sources. A generalized flow of the processes used in reducing GPS baselines is outlined below with its corresponding flowchart shown in figure 10

1. Field observations
2. Download/Import Raw GPS Data from Receivers/Data Storage Media
3. Download Precise Ephemeris Data
4. Pre-Process (Edit and make changes to raw GPS Data if necessary)
5. Set the processing style & baseline flow sequence
6. Process Baseline(s)
7. Evaluate Baseline Reduction Results
8. Make Changes and Rejections

9. Reprocess Baselines and Re-evaluate Results

10. Check Loop Closures and Adjust Baseline Network

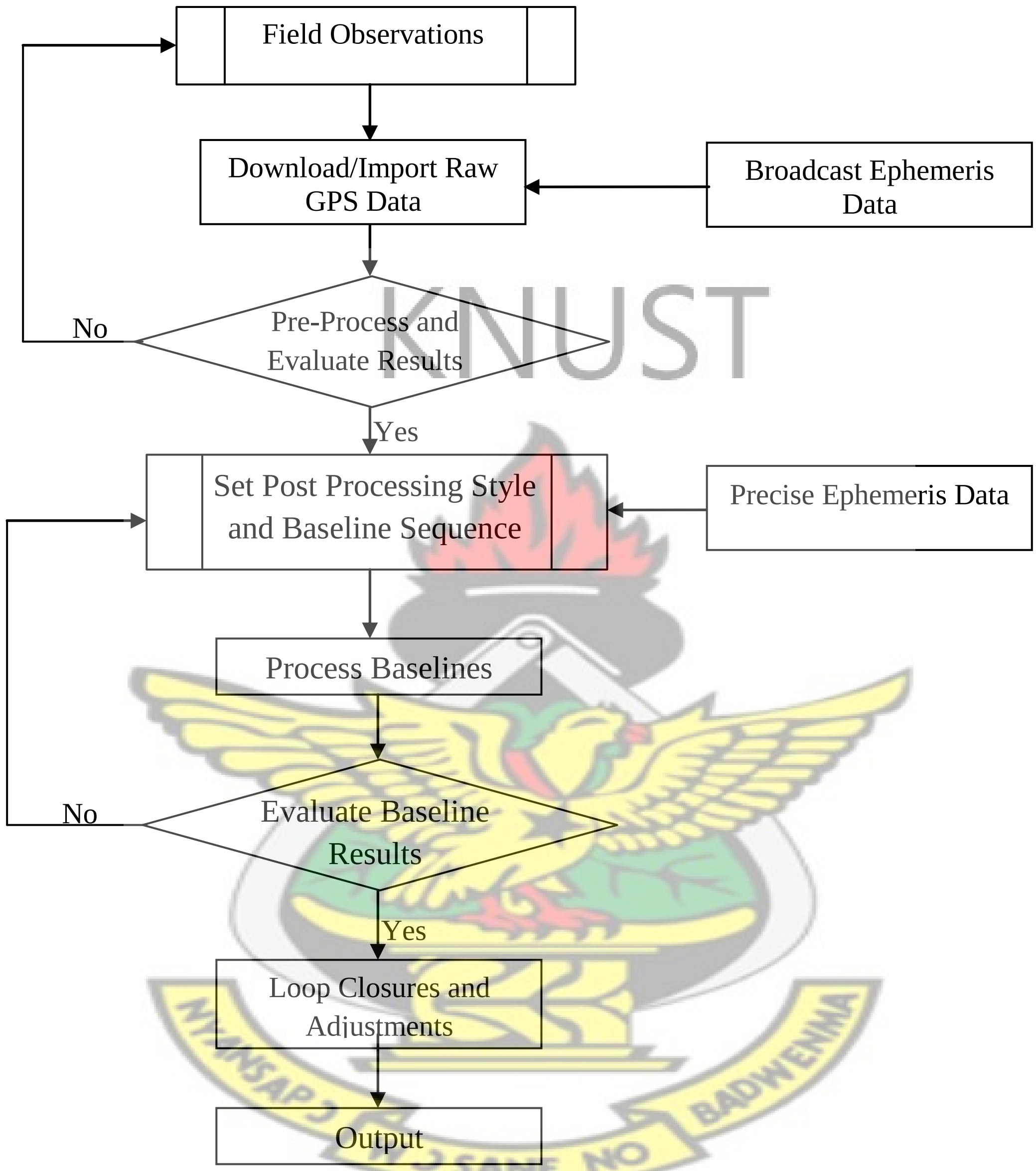

Figure 11 : Data Processing Routine

4.3 Network Adjustment

4.3.1 Network Preadjustment Data Analysis

Prior to adjusting GPS networks, a series of procedures should be followed to analyze the data for internal consistency and to eliminate possible blunders (Ghilani, C. D. and Wolf, P. R., 2006). No control points are needed for these analyses. Depending on the actual observations taken and the network geometry, these procedures may consist of analyzing (1) differences between fixed and observed baseline components, (2) differences between repeated observations of the same baseline components, and (3) loop closures. For the purpose of this project, a loop closure analysis was employed.

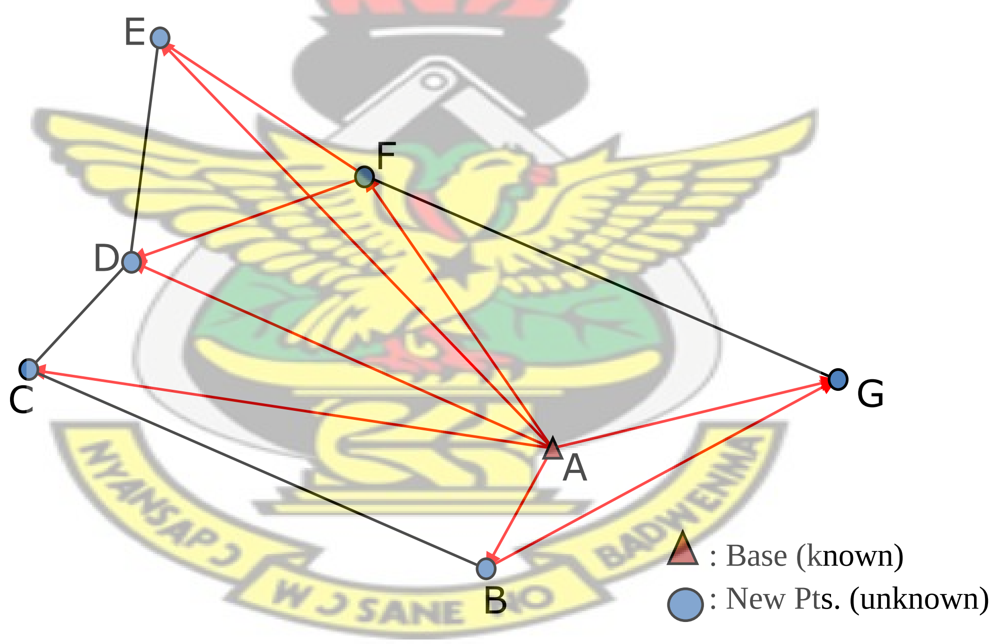

Figure 12: Network Diagram

4.3.2 Analysis of Loop Closures

The GPS network in figure (11) consists of many interconnected closed loops. For example, a closed loop is formed by points *ABGA*. Similarly, *ADFA*, *AEFA*, and so on, are other closed loops. For each closed loop, the algebraic sum of the *X* components should equal zero. The same condition should exist for the *Y* and *Z* components. These loop misclosure conditions are very similar to the leveling loop misclosures imposed in differential leveling and latitude and departure misclosures imposed in closed-polygon traverses. An unusually large misclosure within any loop will indicate that either a blunder or a large random error exists in one (or more) of the baselines of the loop. To compute loop misclosures, denoted as cx, cy and cz, the baseline components are simply added algebraically for the loop chosen. The closures in X, Y & Z for the individual sessions are

$$cx_1 = \Delta X_{AB} + \Delta X_{BG} + \Delta X_{GA} \quad \ldots\ldots\ldots\ldots \quad 4.1$$

$$cy_1 = \Delta y_{AB} + \Delta y_{BG} + \Delta y_{GA} \quad \ldots\ldots\ldots\ldots \quad 4.2$$

$$cz_1 = \Delta z_{AB} + \Delta z_{BG} + \Delta z_{GA} \quad \ldots\ldots\ldots\ldots \quad 4.3$$

$$cx_2 = \Delta X_{AD} + \Delta X_{DF} + \Delta X_{FA} \quad \ldots\ldots\ldots\ldots \quad 4.4$$

$$cy_2 = \Delta y_{AD} + \Delta y_{DF} + \Delta y_{FA} \quad \ldots\ldots\ldots\ldots \quad 4.5$$

$$cz_2 = \Delta z_{AD} + \Delta z_{DF} + \Delta z_{FA} \quad \ldots\ldots\ldots\ldots \quad 4.6$$

$$cx_3 = \Delta X_{AE} + \Delta X_{EF} + \Delta X_{FA} \quad \ldots\ldots\ldots\ldots \quad 4.7$$

$$cy_3 = \Delta y_{AE} + \Delta y_{EF} + \Delta y_{FA} \quad \ldots\ldots\ldots\ldots \quad 4.8$$

$$cz_3 = \Delta z_{AE} + \Delta z_{EF} + \Delta z_{FA} \quad \ldots\ldots\ldots\ldots \quad 4.9$$

Solutions from the equations above are used in computing for the resultant closures for the loops in each session. The resultant closure is given by the equation shown below:

$$\boldsymbol{Resultant\ Closure\ (ce)} = \sqrt{(\boldsymbol{cx^2} + \boldsymbol{cy^2} + \boldsymbol{cz^2})}\ .\ .\ .\ .\ .\ .\ .\ 4.10$$

For evaluation purposes, loop misclosures are expressed in terms of the ratios of resultant misclosures to the total loop lengths. They are given in part per million (ppm).

$$\boldsymbol{ce} = \sqrt{\boldsymbol{cx^2} + \boldsymbol{cy^2} + \boldsymbol{cz^2}}.\ .\ .\ .\ .\ .\ .\ 4.11$$

$$ppm = \frac{ce}{Loop\ length}\ X\ 1{,}000{,}000.\ .\ .\ .\ .\ .\ .\ 4.12$$

Where ***ce*** is the Loop closure error.

The results of the closures and ppm values, as shown by table (4) below, indicate a high level of consistency in the observations. It further indicates that there were neither possible blunders nor random errors in any of the measured loops. Also, comparing ppm ratios against values given in the FGCS guidelines in table (1), it shows that the measurements meet the required accuracy for first order networks.

Table 4 : Loop Closures

ΔABG				
Baseline	ΔX	ΔY	ΔZ	Distance
A - B	142.881	-1430.680	-3687.376	3957.7774
A - G	745.715	4915.739	1493.760	5191.5219
B - G	602.879	6346.415	5181.155	8214.9141
Vector Sum	-0.045	0.004	-0.019	17364.2134
Resultant Closure = 0.049010203 ppm = 2.8224833				
ΔADF				
Baseline	ΔX	ΔY	ΔZ	Distance
A – D	-2367.218	-9704.195	4510.524	10959.9247
A – F	-1557.620	-4022.615	6930.908	8163.6448
F – D	-809.623	-5681.548	-2420.344	6228.4462
Vector Sum	0.025	-0.032	-0.04	25352.0157
Resultant Closure = 0.057 ppm = 2.2483419				
ΔAEF				
Baseline	ΔX	ΔY	ΔZ	Distance
A – E	-2840.489	-8547.794	10659.775	13955.7860
A – F	-1558.959	-4022.089	6931.043	8163.7559
F - E	-1281.535	-4525.663	3728.710	6002.2692
Vector Sum	0.005	-0.042	0.022	28121.8111
Resultant Closure = 0.04767599 ppm = 1.6953385				
AC				
Baseline	ΔX	ΔY	ΔZ	Distance
A – C	-2516.494	-12076.666	2405.430	12568.4007

4.3.3 Least Squares Adjustment

4.3.3.1 Observation Equations

As noted earlier, because GPS networks contain redundant observations, they must be adjusted to make all coordinate differences consistent. In applying least squares to the problem of adjusting baselines in GPS networks, observation equations are written that relate station coordinates to the coordinate differences observed and their residual errors. Observation equations for each measured baseline component are given below: Note that Point A is the fixed point and is used only for check but not used in the adjustment.

$$\partial_{XB} = [X_A + \Delta X_{AB} - X_B] \quad . \quad . \quad . \quad . \; 4.13$$

$$\partial_{YB} = [Y_A + \Delta Y_{AB} - Y_B] \quad . \quad . \quad . \quad . \; 4.14$$

$$\partial_{ZB} = [Z_A + \Delta Z_{AB} - Z_B] \quad . \quad . \quad . \quad . \; 4.15$$

$$\partial_{XC} = [X_A + \Delta X_{AC} - X_C] \quad . \quad . \quad . \quad . \; 4.16$$

$$\partial_{YC} = [Y_A + \Delta Y_{AC} - Y_C] \quad . \quad . \quad . \quad . \; 4.17$$

$$\partial_{ZC} = [Z_A + \Delta Z_{AC} - Z_C] \quad . \quad . \quad . \quad . \; 4.18$$

$$\partial_{XD} = [X_A + \Delta X_{AD} - X_D] \quad . \quad . \quad . \quad . \; 4.19$$

$$\partial_{YD} = [Y_A + \Delta Y_{AD} - Y_D] \quad . \quad . \quad . \quad . \; 4.20$$

$$\partial_{ZD} = [Z_A + \Delta Z_{AD} - Z_D] \quad . \quad . \quad . \quad . \; 4.21$$

$$\partial_{XE} = [X_A + \Delta X_{AE} - X_E] \quad . \quad . \quad . \quad . \; 4.22$$

$$\partial_{YE} = [Y_A + \Delta Y_{AE} - Y_E] \quad . \quad . \quad . \quad . \; 4.23$$

$$\partial_{ZE} = [Z_A + \Delta Z_{AE} - Z_E] \quad . \quad . \quad . \quad . \; 4.24$$

$$\partial_{XF} = [X_A + \Delta X_{AF} - X_F] \quad . \quad . \quad . \quad . \; 4.25$$

$$\partial_{YF} = [Y_A + \Delta Y_{AF} - Y_F] \quad . \quad . \quad . \quad . \; 4.26$$

$$\partial_{ZF} = [Z_A + \Delta Z_{AF} - Z_F] \quad . \quad . \quad . \quad . \; 4.27$$

$$\partial_{XG} = [X_A + \Delta X_{AG} - X_G] \quad . \quad . \quad . \quad . \; 4.28$$

$$\partial_{YG} = [Y_A + \Delta Y_{AG} - Y_G] \quad . \quad . \quad . \quad . \; 4.29$$

$$\partial_{ZG} = [Z_A + \Delta Z_{AG} - Z_G] \quad . \quad . \quad . \quad . \; 4.30$$

$$\partial_{XG} - \partial_{XB} = [X_B + \Delta X_{BG} - X_G] \quad . \quad . \; 4.31$$

$$\partial_{YG} - \partial_{YB} = [Y_B + \Delta Y_{BG} - Y_G] \quad . \quad . \; 4.32$$

$$\partial_{ZG} - \partial_{ZB} = [Z_A + \Delta Z_{BG} - Z_G] \quad . \quad . \; 4.33$$

$$\partial_{XD} - \partial_{XF} = [X_F + \Delta X_{FD} - X_D] \quad . \quad . \; 4.34$$

$$\partial_{YD} - \partial_{YF} = [Y_F + \Delta Y_{FD} - Y_D] \quad . \quad . \; 4.35$$

$$\partial_{ZD} - \partial_{ZF} = [Z_F + \Delta Z_{FD} - Z_D] \quad . \quad . \; 4.36$$

$$\partial_{XE} - \partial_{XF} = [X_F + \Delta X_{FE} - X_E] \quad . \quad . \; 4.37$$

$$\partial_{YE} - \partial_{YF} = [Y_F + \Delta Y_{FE} - Y_E] \quad . \quad . \; 4.38$$

$$\partial_{ZE} - \partial_{ZF} = [Z_F + \Delta Z_{FE} - Z_E] \quad . \quad . \; 4.39$$

Observation equations of the foregoing form would be written for all measured baselines in any figure. For Figure (11), a total of 9 baselines were observed, so the number of observation equations that can be developed is 27. Also, stations *B, C, D, E, F and G* each have three unknown coordinates, for a total of 18 unknowns in the problem. Thus, there are 27 - 18 = 9 redundant observations in the network. The 27 observation equations can be expressed in the form $A_x = L + V$.

Where A is the coefficient matrix (A is m $\times$ n with m > n)

L is the matrix of absolute terms

V is the residuals (errors)

X is the matrix of unknown parameters (least-squares solution)

4.3.3.2 Normal Equation (N)

$A_X = L + V$

The above equation can be normalized by pre multiplying by $(A^T W)$ to obtain

$(A^T WA)_X = (A^T WL) + (A^T WV)$, but $(A^T WV) = 0$, so

This reduces to the form $N_X = B$ - - - - - Normal equation.

$X = (A^T WA)^{-1}.\ (A^T WL)$ - - - - $X = N^{-1} B$ 4.40

Reference Standard deviation (δ_0) = $\sqrt{\frac{V^T WV}{m-n}}$ 4.41

Where m = number of equations &

n = number of unknowns.

The covariance matrix ($\sum$) = $\delta_0^2 (A^T WA)^{-1}$ - - - $\delta_0^2 N^{-1}$ 4.42

Standard deviation of adjusted quantities (σ) = $\delta_0 \sqrt{\sum}$ 4.43

Standard Errors $(\sigma_{\hat{X}}) = \sqrt{diag(\sum)}$ 4.44

Matrices formed from the observation equations are shown below:

Design Matrix (A)

$$
\begin{vmatrix}
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \\
-1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\
0 & -1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 \\
0 & 0 & -1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \\
0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & -1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & -1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & -1 & 0 & 0 & 0
\end{vmatrix}
$$

$$L = \begin{vmatrix} 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ 0.04500 \\ -0.00400 \\ 0.01900 \\ -1.36400 \\ 0.55800 \\ 0.17500 \\ -0.00500 \\ 0.04200 \\ -0.02200 \end{vmatrix} \quad X = \begin{vmatrix} \partial XB \\ \partial YB \\ \partial ZB \\ \partial XC \\ \partial YC \\ \partial ZC \\ \partial XD \\ \partial YD \\ \partial ZD \\ \partial XE \\ \partial YE \\ \partial ZE \\ \partial XF \\ \partial YF \\ \partial ZF \\ \partial XG \\ \partial YG \\ \partial ZG \end{vmatrix} \quad X = \begin{vmatrix} XB \\ YB \\ ZB \\ XC \\ YC \\ ZC \\ XD \\ YD \\ ZD \\ XE \\ YE \\ ZE \\ XF \\ YF \\ ZF \\ XG \\ YG \\ ZG \end{vmatrix} \quad V = \begin{vmatrix} v_{XAB} \\ v_{yAB} \\ v_{zAB} \\ v_{XAC} \\ v_{YAC} \\ v_{ZAC} \\ v_{XAD} \\ v_{YAD} \\ v_{ZAD} \\ v_{XAE} \\ v_{YAE} \\ v_{ZAE} \\ v_{XAF} \\ v_{YAF} \\ v_{ZAF} \\ v_{XAG} \\ v_{YAG} \\ v_{ZAG} \\ v_{XBG} \\ v_{YBG} \\ v_{ZBG} \\ v_{XFD} \\ v_{YFD} \\ v_{ZFD} \\ v_{XFE} \\ v_{YFE} \\ v_{ZFE} \end{vmatrix}$$

4.3.3.3 The Covariance and Weight Matrices

Note that the observation equations for GPS network adjustment are linear and that the only nonzero elements of the *A* matrix are either 1 or -1. In GPS relative positioning, the three observed baseline components are correlated. Therefore, a 3 X 3 covariance matrix is derived for each baseline as a product of the least squares adjustment of the carrier-phase measurements. (Ghilani, C. D. and Wolf, P. R, 2006 ; Cross, P. A, 1994). This covariance matrix is used to properly weight the observations in the network adjustment. The weight matrix for any GPS network is therefore a block-diagonal type, with an individual 3 X 3 matrix for each baseline observed on the diagonal to provide the correlation that exists between baselines observed simultaneously.

The Covariance ($\sum$) amd Weight (W) Matrices for baselines of the network in figure (11) are therefore of the form:

$$\sum_{AB} = \begin{bmatrix} \sigma_x^2 & \sigma_{xy} & \sigma_{xz} \\ \sigma_{xy} & \sigma_y^2 & \sigma_{yz} \\ \sigma_{xz} & \sigma_{yz} & \sigma_z^2 \end{bmatrix} \quad and \quad W = \sum_{AB}{}^{-1}$$

The complete covariance and weight matrices for the network of Figure (11) both have dimensions of 27 X 27.

$$V = \begin{vmatrix} -0.0060421 \\ 0.0005442 \\ -0.0023762 \\ 0.0000000 \\ 0.0000000 \\ 0.0000000 \\ -0.2105067 \\ 0.0673771 \\ 0.0481329 \\ 0.0570996 \\ -0.0208252 \\ -0.0152706 \\ 0.1764002 \\ -0.1484503 \\ -0.0295566 \\ 0.0137849 \\ -0.0013463 \\ 0.0063130 \\ -0.0251730 \\ 0.0021095 \\ -0.0103108 \\ 0.9770931 \\ -0.3421726 \\ -0.0973106 \\ -0.1143006 \\ 0.0856251 \\ 0.0362860 \end{vmatrix}$$

$$X_0 = \begin{vmatrix} 6229819.2190 \\ -1178825.6710 \\ 691182.9910 \\ 6227159.8440 \\ -1189471.6570 \\ 697275.7970 \\ 6227309.1200 \\ -1187099.1860 \\ 699380.8910 \\ 6226835.8490 \\ -1185942.7850 \\ 705530.1420 \\ 6228117.3790 \\ -1181417.0800 \\ 701801.4100 \\ 6230422.0530 \\ -1172479.2520 \\ 696364.1270 \end{vmatrix} \quad \hat{X} = \begin{vmatrix} -0.00600 \\ 0.00050 \\ -0.00240 \\ 0.00000 \\ 0.00000 \\ 0.00000 \\ -0.21050 \\ 0.06740 \\ 0.04810 \\ 0.05710 \\ -0.02080 \\ -0.01530 \\ 0.17640 \\ -0.14850 \\ -0.02960 \\ 0.01380 \\ -0.00130 \\ 0.00630 \end{vmatrix} \quad X = \begin{vmatrix} 6229819.21296 \\ -1178825.67046 \\ 691182.98862 \\ 6227159.84400 \\ -1189471.65700 \\ 697275.79700 \\ 6227308.90949 \\ -1187099.11862 \\ 699380.93913 \\ 6226835.90610 \\ -1185942.80583 \\ 705530.12673 \\ 6228117.55540 \\ -1181417.22845 \\ 701801.38044 \\ 6230422.06678 \\ -1172479.25335 \\ 696364.13331 \end{vmatrix}$$

X_0 provisional coordinates

$\hat{X}$ Adjusted/estimated parameters

X Adjusted observations ($X_0 + \hat{X}$)

(δ_0^2) The unit variance = **0.002614**

SE The standard errors

$$SE = \begin{vmatrix} 0.0086749 \\ 0.0034785 \\ 0.0030440 \\ 0.0151477 \\ 0.0075833 \\ 0.0063471 \\ 0.0136773 \\ 0.0052623 \\ 0.0049574 \\ 0.0120754 \\ 0.0057414 \\ 0.0047202 \\ 0.0129618 \\ 0.0072684 \\ 0.0059740 \\ 0.0121934 \\ 0.0057936 \\ 0.0044202 \end{vmatrix}$$

From (Thomas, 1976) and (Mikhail, 1976), the above errors in (X, Y, Z) can be propagated to (ϕ, λ, h) using Gauss Law of Propagation of errors, defined by the equations below.

$$C_{(x,y,z)} = AC_{(\phi,\lambda,h)}A^T$$

$$C_{(\phi,\lambda,h)} = A^{-1}C_{(x,y,z)}(A^{-1})^T$$

4.3.3.4 Unit variance Test

δ_0^2 = 0.002614

δ_0 = 0.051127292

$\delta_0 < 1$, so

F = $1/\delta_0^2$ = 382.55547

$V_1 = \infty$, V_2 = (m-n) = 9

At a 95% (α = 0.05), 0.025(i.e. $\alpha/2$) level of confidence the F table gives, for $V_1 = \infty$ and V_2 = 9, a percentile of 2.71. So computing F from the above equation as 382.55547 leads to the conclusion that the computed value for the unit variance is significantly different from unity, therefore, the assigned a priori covariance matrix was multiplied by δ_0^2 (Cross, 1994).

5.0 CHAPTER FIVE - RESULTS AND ANALYSIS

This chapter presents the results obtained from the processing and adjustment processes from chapter four (4). Results obtained from each stage of the previous chapter are presented into tables as shown below. A brief discussion explaining the results in each table follows immediately.

Table 5 : Processed UTM & Geographic Coordinates of Camp Ramrod (WGS84)

Stn. ID	UTM Zone 29 Coordinates		Geographic Coordinates		Elevation (m)	Stn. Location
	Eastings (m)	Nothings (m)	Latitude	Longitude		
RAMR^B	311658.554	696294.400	6°17' 47.6257'' N	10° 42' 09.1246'' W	46.533	72nd Barracks Monrovia

Table 6 : UTM & Geographic coordinates of Camp Ramrod given by DLSC

Stn. ID	UTM Zone 29 Coordinates		Geographic Coordinates		Elevation (m)	Stn. Location
	Eastings (m)	Nothings (m)	Latitude	Longitude		
RAMR^B	311630.8400	696206.8200	6°17' 46.8480'' N	10°42' 09.9120'' W	16.0949	72nd Barracks Monrovia

Table 7 : Difference between the Coordinates of Camp Ramrod as presented in Tables 5&6

Stn. ID	UTM Zone 29 Coordinates		Geographic Coordinates		Elevation (m)	Stn. Location
	Eastings (m)	Nothings (m)	Latitude	Longitude		
RAMR^B	27.714	87.58	0°00' 00.7777'' N	-0°00' 00.7874'' W	30.438	72nd Barracks Monrovia

As earlier stated, we could not rely on the data provided us by DLSC for the reference station at 72nd Community for two simple reasons: firstly, DLSC told us that original data for the old network was destroyed during the long years of war. Secondly, DLSC could not provide us information on how and by whom was the data they were giving us generated.

The huge disparity between the data given and the one generated by means of PPP, as shown in table 7 above, is sufficient justification for our initial fears of using the given data. Though the geographic coordinates from the two datasets compare relatively close, the unacceptable difference between their UTM coordinates show that the projected Northing coordinate was in error. Also, neither the duration of observation nor the epoch to which the original data refers is known; hence the fear of not using the given data was justified. The results shown in Table 6 have been used as the known coordinates of the base station for the computation of differential corrections for all sessions tied to the reference station at 72nd Community. The next two tables that follow show the pre and post-processed results of the UTM coordinates of new network points.

Table 8 : Pre-processed UTM Coordinates

Pre-processed UTM Coordinates				
Stn. ID	Eastings (m)	Northings(m)	Height (m)	Stn. Location
A	311657.7288	696292.7579	53.141	72nd Barracks Monrovia
B	310266.9753	692590.2947	47.7372	TB_Annex, Monrovia
C	299331.1007	698749.0448	102.4702	Ducor Hill, Monrovia
D	301696.614	700869.0257	30.7311	Freeport P. Stn. Monrovia
E	302766.0457	707049.6603	43.8431	St. Paul Bridge Monrovia
F	307438.6447	703281.7499	40.7687	Goodridge, Monrovia
G	316632.7232	697784.5382	35.1009	Omega Tower, Monrovia

Table 9 : Post-processed UTM Coordinates

Post-processed UTM Coordinates				
Stn. ID	Eastings (m)	Northings(m)	Height (m)	Stn. Location
A	311658.554	696294.400	46.533	72nd Barracks Monrovia
B	310267.193	692589.105	46.994	TB_Annex, Monrovia
C	299331.916	698750.940	94.16	Ducor Hill, Monrovia
D	301698.167	700867.986	30.772	Freeport P. Stn. Monrovia
E	302767.963	707051.609	34.309	St. Paul Bridge Monrovia
F	307439.319	703283.605	33.072	Goodridge, Monrovia
G	316632.287	697782.822	33.557	Omega Tower, Monrovia

It is understandable and acceptable that differences exist between the pre-processed and post-processed coordinates of new network stations as shown by tables 8 and 9 above.

Pre-processing was done in Liberia using broadcast ephemeris which produces results of relatively lower accuracies. Field data was pre-processed immediately after data collection. Pre-processing was carried out for the sole purpose of judging if the data collected was of adequate quality for this project, and to make decisions as to the need for any re-observations or revisions of plans.

Post-processing took place in Ghana using precise ephemeris data downloaded from the internet. The data for the base station was also converted to RINEX format and sent for online processing in Canada thus refining the solutions of the post-processed results.

The following two tables show the results of the unadjusted and adjusted Cartesian coordinates of new network stations.

Table 10 : Result of Unadjusted Cartesian Coordinates

	Unadjusted Cartesian Coordinates			
Stn. ID	X	Y	Z	Stn. Location
B	6229819.219	-1178825.671	691182.991	TB_Annex, Monrovia
C	6227159.844	-1189471.657	697275.797	Ducor Hill, Monrovia
D	6227309.120	-1187099.186	699380.891	Freeport P. Stn. Monrovia
E	6226835.849	-1185942.785	705530.142	St. Paul Bridge Monrovia
F	6228117.379	-1181417.080	701801.410	Goodridge, Monrovia
G	6230422.053	-1172479.252	696364.127	Omega Tower, Monrovia

Table 11 : Result of Adjusted Cartesian Coordinates

		Adjusted Cartesian Coordinates		
Stn. ID	X	Y	Z	Stn. Location
B	6229819.213	-1178825.671	691182.989	TB_Annex, Monrovia
C	6227159.844	-1189471.657	697275.797	Ducor Hill, Monrovia
D	6227308.910	-1187099.119	699380.939	Freeport P. Stn. Monrovia
E	6226835.906	-1185942.806	705530.127	St. Paul Bridge Monrovia
F	6228117.555	-1181417.228	701801.380	Goodridge, Monrovia
G	6230422.067	-1172479.253	696364.133	Omega Tower, Monrovia

It can be seen from the tables above (10 and 11) that differences between the unadjusted and adjusted Cartesian coordinates exists only in the decimal part of the coordinates. This further confirms the pre-adjustment test (loop closure) results, indicating a higher level of consistency of the measurements. These differences, no matter how small, point to the fact that rigorous least square adjustment is necessary for any serious geodetic work.

These differences exist not only with Cartesian coordinates, but also with UTM and Geographic coordinates, as shown in the table below (12).

Table 12 : Result of Adjusted UTM & Geographic Coordinates

Stn. ID	UTM Zone 29 Coordinates		Geographic Coordinates		Elevation (m)	Stn. Location
	Eastings (m)	Nothings (m)	Latitude (DMS) N	Longitude (DMS) W		
B	310267.1920	692589.1030	6°15' 46.864"	10° 42'53.995"	46.9870	TB_Annex, Monrovia
C	299331.9150	698750.9400	6° 19'06.240"	10° 48'50.400"	94.1600	Ducor Hill, Monrovia
D	301697.9010	700868.0710	6° 20' 15.415"	10° 47' 33.678"	29.7750	Freeport P. Stn. Monrovia
E	302766.7430	707051.7400	6° 23' 36.816"	10° 46' 59.598"	34.5810	St. Paul Bridge Monrovia
F	307439.5540	703283.6850	6°21' 34.689"	10° 44' 27.141"	31.8690	Goodridge, Monrovia
G	316632.2650	697782.7230	6° 18' 36.595"	10° 39' 27.467"	33.4170	Omega Tower, Monrovia

The following tables (13, 14 & 15) present the results of adjusted Cartesian, UTM and Geographic coordinates along with their standard errors (SE) of measurement.

Table 13 : Result of Adjusted Cartesian Coordinates with their standard Errors

Stn. ID	Adjusted Cartesian Coordinates and their Standard Errors					
	X (m)	SE (m)	Y (m)	SE (m)	Z (m)	SE (m)
B	6229819.2132	*0.008675*	-1178825.6706	*0.003478*	691182.9887	*0.003044*
C	6227159.8440	*0.001514*	-1189471.6570	*0.007583*	697275.7970	*0.006347*
D	6227308.9633	*0.001367*	-1187099.1282	*0.005262*	699380.9055	*0.004957*
E	6226835.8685	*0.001207*	-1185942.7893	*0.005741*	705530.1360	*0.004720*
F	6228117.4585	*0.001296*	-1181417.1484	*0.007268*	701801.4012	*0.005974*
G	6230422.0682	*0.001219*	-1172479.2533	*0.005794*	696364.1338	*0.004420*

Table 14 : Result of Adjusted UTM Coordinates with their standard errors

Stn. ID	Eastings (m)	SE (m)	Northings (m)	SE (m)	Height (m)	SE (m)
B	310267.1920	*0.008675*	692589.1030	*0.003478*	46.9870	*0.003044*
C	299331.9150	*0.001514*	698750.9400	*0.007583*	94.1600	*0.006347*
D	301697.9010	*0.001367*	700868.0710	*0.005262*	29.7750	*0.004957*
E	302766.7430	*0.001207*	707051.7400	*0.005741*	34.5810	*0.004720*
F	307439.5540	*0.001296*	703283.6850	*0.007268*	31.8690	*0.005974*
G	316632.2650	*0.001219*	697782.7230	*0.005794*	33.4170	*0.004420*

Table 15 : Result of Adjusted Geographic Coordinates with their standard errors

Stn. ID	Latitude (DMS) N	SE (m)	Longitude (DMS) W	SE (m)	Height (m)	SE (m)
B	6° 15' 46.864"	*0.008675*	10° 42' 53.995"	*0.003478*	46.9870	*0.003044*
C	6°19' 06.240"	*0.001514*	10° 48' 50.400"	*0.007583*	94.1600	*0.006347*
D	6° 20' 15.415"	*0.001367*	10° 47' 33.678"	*0.005262*	29.7750	*0.004957*
E	6° 23' 36.816"	*0.001207*	10° 46' 59.598"	*0.005741*	34.5810	*0.004720*
F	6° 21' 34.689"	*0.001296*	10° 44' 27.141"	*0.007268*	31.8690	*0.005974*
G	6° 18' 36.595"	*0.001219*	10° 39' 27.467"	*0.005794*	33.4170	*0.004420*

All the computed values for standard errors shown in Tables 13, 14 & 15 are less than 1cm, and this value clearly indicates a strong level of consistency in data collected. The results obtained from the pre-adjustment analyses also indicate a great measure of consistency and no possible blunder in observation sessions conducted. These indications justify the reliability of the final results obtained from the full-scale least squares adjustment.

Unlike standard deviation which is more or less constant, standard error largely depends on the number of measurements; the larger the measurements, the smaller the standard error and vice versa.

Below is the diagram of the new network, overlaid on the topographic map of the project area.

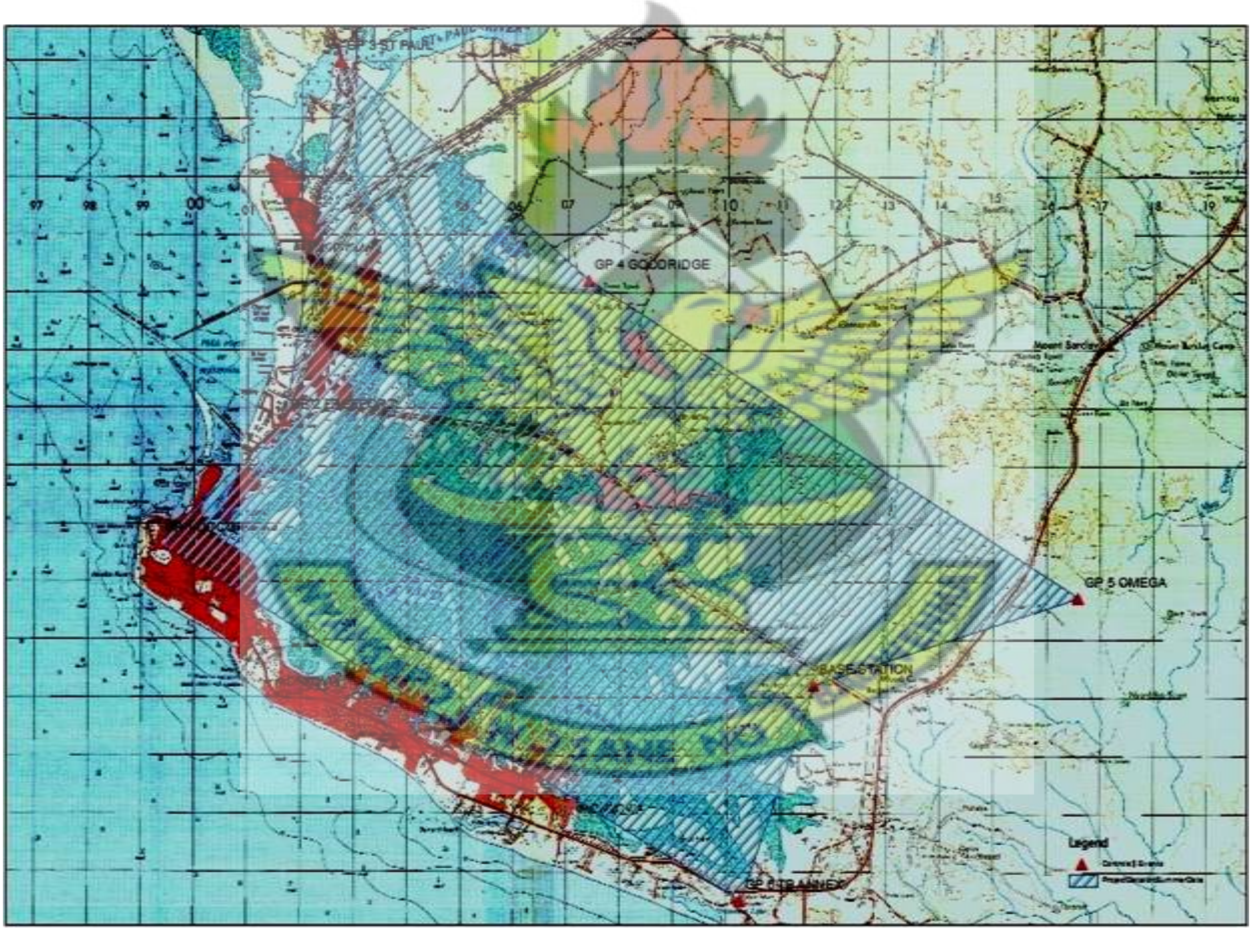

Figure 13: New Network Diagram

The above project diagram, overlaid on the topographic map of the project area, fits the new points exactly on their true ground positions as they appear on this map. This further confirms the level of accuracy and reliability of the measurements.

6.0 CHAPTER SIX – CONCLUSIONS AND RECOMMENDATION

In this project, a first order geodetic reference network of six (6) primary controls has been established in Montserrado County-Liberia using DGPS. This network will be accessible by everyone without any charges. It will also serve as a pilot project for the future densification of the National Geodetic Network.

Six (6) pre-cast reinforced concrete beacons were solidly installed and cemented into the ground at various points within the project area. Three (3) ten (10)-hour observation sessions were carried out at the base station in 72nd Community and the data was used to compute its coordinates using the method of PPP. This point then served as a fixed point within the network and was used to compute GPS baselines to other points for the purpose of coordinating them.

Point description cards were provided for all points in the network containing UTM Coordinates, satellite images, ground photos and oral descriptions for easy access.

With the network now established and final coordinates derived, it is clearly seen that all stated objectives were attained and the following conclusions have been drawn.

6.1 Conclusion

It is obvious from the preceding chapters that a reliable geodetic reference network is a basic requirement for the successful execution of all survey related projects. It is the wire-frame or the skeleton on which continuous and consistent mapping, Geographic Information Systems (GIS), and surveys are based. Locating spatial features with respect to geodetic control enables the accuracy assessment of these features. Interest and activity regarding geodetic control has dramatically increased at all government levels in Liberia because of the need for accurate maps and surveys needed for national development and environmental aplications. Unfortunately, such a network is lacking in Liberia as the only one that existed before the war now lie in ruins in the aftermath of the nearly two (2) decades of war. The established network will now serve those basic needs and remedy some serious problems posed by the lack of a geodetic network in Montserrado County.

The new network provides coordinates in the WGS84 system to be transformed to the Liberian local system when transformation parameters are available. It will reduce, if not eliminate, the use of compass survey and the metes and bounds method for property location and demarcation as the new network provides reference stations established with sub-centimeter accuracy.

The establishment of this network will encourage the use of GNSS technology in general, and GPS in particular, by Liberian surveyors in the execution of their daily surveying duties. This will enhance accuracy and reduce numerous land conflicts resulting from poorly demarcated boundaries established by means of compass.

6.2 Recommendations

From the execution of this project, the following recommendations regarding upgrading, the use and maintenance of this network are made:

1. The network should be renamed to conform to the DLSC naming standard/system.
2. All network stations should be fenced and/or protected to ensure their safety and security.
3. Longer observation sessions should be observed on all the stations to compare and/or improve their final coordinates for integration into the national system. This is necessary as concrete beacons undergo a longer period of curing and settlement.

4. These stations should be developed further into CORS stations.

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7.2 APPENDIX A : Point Description Cards

POINT DESCRIPTION CARD	
Point Name	GP1-1 Ducor Hill
Point Location	Masonic Temple Yard
Type of Monument	Concrete beacon with brass cap/marker
Date Established	April 04, 2013
Established by	C. Sylvester N. Bundoo & Mahmoud Solomon

UTM (Zone 29) Coordinates		Cartesian Coordinates	
E (m)	299331.9150	X (m)	6227159.8440
N (m)	698750.9400	Y (m)	-1189471.6570
H (m)	94.1600	Z (m)	697275.7970

Satellite Image	Ground Photo
	

Description

The point is located on the premises of the Masonic Temple up Benson street. It is situated 15 ft. from the base of the memorial candle monument, on the side that faces the Masonic building. The point is a concrete beacon that rises 1 ft. above ground with a circular brass cap at the center of its top bearing the inscription; Liberia Ministry of Lands, Mines, and Energy; Survey Marker; "Do not disturb".

POINT DESCRIPTION CARD	
Point Name	GP1-2 Freeport
Point Location	Freeport Police Station Yard
Type of Monument	Concrete beacon with brass cap/marker
Date Established	April 03, 2013
Established by	C. Sylvester N. Bundoo & Mahmoud Solomon

UTM (Zone 29) Coordinates		Cartesian Coordinates	
E (m)	301697.9010	X (m)	6227308.9095
N (m)	700868.0710	Y (m)	-1187099.1186
H (m)	29.7750	Z (m)	699380.9391

Satellite Image

Ground Photo

Description

The point is located on the premises of the Freeport Police Station on Bushrod Island. It is situated about 20 ft. in front of the Police Station Building toward Clara town and the Somalia Drive-end of the junction. The point is a concrete beacon that rises 1 ft. above ground with a circular brass cap at the center of its top bearing the inscription; Liberia Ministry of Lands, Mines, and Energy; Survey Marker; “Do not disturb”.

POINT DESCRIPTION CARD

Point Name	GP1-3 St. Paul Bridge
Point Location	Duala-end of St. Paul Bridge
Type of Monument	Concrete beacon with brass cap/marker
Date Established	April 03, 2013
Established by	C. Sylvester N. Bundoo & Mahmoud Solomon

UTM (Zone 29) Coordinates		Cartesian Coordinates	
E (m)	302766.7430	X (m)	6226835.9061
N (m)	707051.7400	Y (m)	-1185942.8058
H (m)	34.5810	Z (m)	705530.1267

Sketch	Topographic Map
	

Description

The point is located on the Monrovia-end of the St. Paul Bridge; between the train track and the asphalt concrete bridge. It is situated just at the edge of the abutment. The point is a concrete beacon that rises 1 ft. above ground with a circular brass cap at the center of its top bearing the inscription; Liberia Ministry of Lands, Mines, and Energy; Survey Marker; “Do not disturb”.

POINT DESCRIPTION CARD

Point Name	GP1-4 Goodridge
Point Location	E. Jonathan Goodridge Campus, Barnesville Est.
Type of Monument	Concrete beacon with brass cap/marker
Date Established	April 04, 2013
Established by	C. Sylvester N. Bundoo & Mahmoud Solomon

UTM (Zone 29) Coordinates		Cartesian Coordinates	
E (m)	307439.5540	X (m)	6228117.5554
N (m)	703283.6850	Y (m)	-1181417.2284
H (m)	31.8690	Z (m)	701801.3804

Sketch	Topographic Map
	

Description

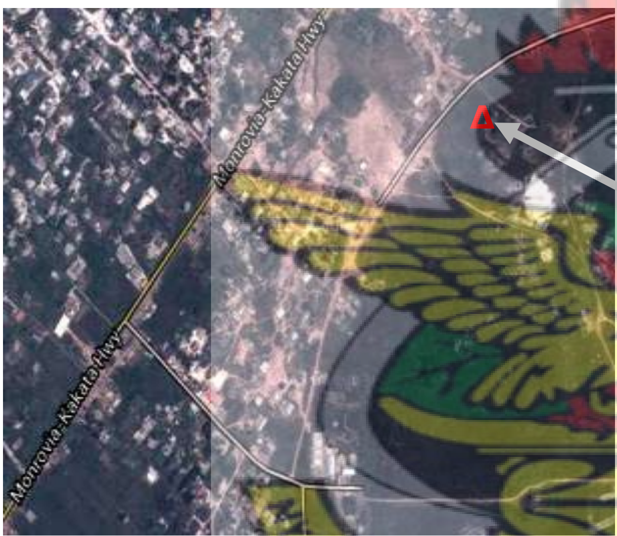
The point is located on the premises of the E. Jonathan Goodridge Public School in Barnesville Estate. It is situated about 25 ft. in front of the school's Building toward the North-eastern (swamp) end of the fence, besides the school's septic tank. The point is a concrete beacon that rises 1 ft. above ground with a circular brass cap at the center of its top bearing the inscription; Liberia Ministry of Lands, Mines, and Energy; Survey Marker; "Do not disturb".

POINT DESCRIPTION CARD

Point Name	GP1-5 Omega
Point Location	Omega Tower Community, Paynesville
Type of Monument	Concrete beacon with brass cap/marker
Date Established	April 08, 2013
Established by	C. Sylvester N. Bundoo & Mahmoud Solomon

UTM (Zone 29) Coordinates		Cartesian Coordinates	
E (m)	316632.2650	X (m)	6230422.0668
N (m)	697782.7230	Y (m)	-1172479.2533
H (m)	33.4170	Z (m)	696364.1333

Sketch	Topographic Map



Description

The point is located on the premises of the formal Omega Tower in Paynesville. It is situated on the Eastern-end of the Monrovia-Kakata Highway. From the main highway, a dusty road branches right for about 20m before joining the tower's perimeter. A circular road runs almost parallel to the main highway toward kakata for about 100m before reaching Mr. John Cooper's residence. The point is directly opposite Mr. Cooper's building, just about 10m on the right-side of the road. The point is a concrete beacon that rises 1 ft. above ground with a circular brass cap at the center of its top bearing the inscription; Liberia Ministry of Lands, Mines, and Energy; Survey Marker; "Do not disturb".

POINT DESCRIPTION CARD

Point Name	GP1-6 TB-Annex
Point Location	TB-Annex Yard, Congo Town Back Road
Type of Monument	Concrete beacon with brass cap/marker
Date Established	April 03, 2013
Established by	C. Sylvester N. Bundoo & Mahmoud Solomon

UTM (Zone 29) Coordinates		Cartesian Coordinates	
E (m)	310267.1920	X (m)	6229819.2130
N (m)	692589.1030	Y (m)	-1178825.6705
H (m)	46.9870	Z (m)	691182.9886

Satellite Image	Ground Photo
	

Description

The point is located on the premises of the T. B. Annex Hospital on Congo Town back road, directly opposite the Ministry of Health and Social Welfare. It is situated about 15 ft. from the main road and 15 ft. on the right of the dusty road that leads to the hospital. The point is a concrete beacon that rises 1 ft. above ground with a circular brass cap at the center of its top bearing the inscription; Liberia Ministry of Lands, Mines, and Energy; Survey Marker; “Do not disturb”.

7.3 APPENDIX B: Field Data Log Form

Field Data Log Sheet		
Date		Remark
Point ID		
Point Location		
Session No.		
Surveyor		
Receiver Sr.No.		
Before measurement		
Time	Antenna hgt.	
After measurement		
Time	Antenna hgt.	
Date		Remark
Point ID		
Point Location		
Session No.		
Surveyor		
Receiver Sr.No.		
Before measurement		
Time	Antenna hgt.	
After measurement		
Time	Antenna hgt.	
Date		Remark
Point ID		
Point Location		
Session No.		
Surveyor		
Receiver Sr.No.		
Before measurement		
Time	Antenna hgt.	
After measurement		
Time	Antenna hgt.	