3-D READJUSTMENT OF PARTS OF THE NIGERIAN PRIMARY TRIANGULATION NETWORK WITH GNSS DATA

BY

OLIVIA CHIDIMMA NWEZE

DEPARTMENT OF GEOMATICS,

FACULTY OF ENVIRONMENTAL DESIGN,

AHMADU BELLO UNIVERSITY,

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Olivia Chidimma NWEZE
BSc. Land Surveying (ATBU), 2010
(P14EVGM8003)

A DISSERTATION SUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES, AHMADU BELLO UNIVERSITY, ZARIA
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DEPARTMENT OF GEOMATICS,
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JUNE, 2019

DECLARATION

I declare that the work in this Dissertation entitled "**3-D readjustment of part of the Nigerian primary triangulation network with GNSS data"** has been carried out by me in the Department of Geomatics. The information derived from literature has been duly acknowledged in the text and a list of references provided. No part of this project was previously presented for another degree or diploma at this or any other institution.

NWEZE, Olivia Chidimma	______________	______________________
(P14EVGM8003)	Signature	Date

CERTIFICATION

This Dissertation entitled "**3-D READJUSTMENT OF PART OF THE NIGERIAN PRIMARY TRIANGULATION NETWORK IN NIGERIA WITH GNSS DATA"** by Olivia Chidimma NWEZE meets the regulations governing the award of Masters of Science (M.Sc.) degree in Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

Dr. J. D. Dodo ______________________ ___________________

(Chairman, Supervisory Committee) Signature Date

Prof. L. M. Ojigi ______________________ ___________________

(Member, Supervisor Committee) Signature Date

Dr. O. A. Isioye ______________________ ___________________

(Head of Department) Signature Date

Prof. S. Z. Abubakar ______________________ ___________________

(Dean, School of Postgraduate Studies) Signature Date

DEDICATION

I dedicate this Dissertation to my mother for her constant, unconditional love, care, advice, prayers and encouragement. I also dedicate it to the loving memory of my beloved Father, Chief Allison OnyedikachiNjoku (November 30, 1945 – September 27, 2009).

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ABSTRACT

Current development has shown many countries improving and strengthening their national control networks using modern space geodetic system. This study carried out 3D readjustment of part of the Nigerian Primary Triangulation Network using GNSS data obtained from Office of the Surveyor General of the Federation (OSGoF). First, the geometric analysis of the existing Nigerian Primary Network were evaluated using Triangle Inequality theorem and on how well the triangles in the network are conditioned. The result revealed that the network fulfilled the condition of the theorem however, when subjected to how well-conditioned the triangles were within the network, it was discovered that 56% of the triangles met the requirement while 44% did not meet the geometric conditionality. Different processing strategies are capable of giving different coordinate solutions for same point. Using fifty-two (52) GNSS station observational campaigns carried out within the period of October, 2010 – April, 2011, the study performed comparative evaluation of three different GNSS post-processing strategies with respect to points of reference originally processed with BERNESSE software from OSGoF. These processing strategies include; reducing observational campaign observed from pairs of stations (baselines) and combining these baselines into a network (Approach 1), taking GPS observations observed simultaneously at all stations directly into a network adjustment where all the coordinates of the network are presents as unknowns (Approach 2) and lastly, processing the observations using Precise point Positioning techniques (Approach 3). Due to the dissimilar nature of positioning, Trimble Total Control software was used to process Approach 1 and 2 solution (Relative solution approach) while GNSS-lab tool (gLAB) was used to process Approach 3 (Stand-alone solution approach). The residual (differences) in the horizontal and vertical component were computed for all observations. Out of the three

solutions, Approach 3 gave solutions that were closest to the points of reference, followed by Approach 1 and then Approach 2. Poor performance of Approach 2 was attributed to some restraining factors that considerably induced errors within its solutions. Improvement on the study will be on how to develop a standard approach for harmonizing GNSS solutions in the near future.

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CHAPTER ONE

INTRODUCTION

1.1 Background to the Study

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 that serve as a common reference framework of control points is hinged on points whose 3D positions are known to a high degree of accuracy permitting the many and varied surveying, mapping and charting programs to be referenced to that common systemfor various types of user (Acheampong**,** 2008).

Geodetic control networks are established by various methods. The classical (traditional) methods are traversing, triangulation and trilateration (Schofield and Breach, 2007). Provision of control points defining these reference frames by these conventional methods are expensive, tedious, limited to intervisibility between beacons and the area of survey, thus reducing the effectiveness of networks at night and in poor weather conditions.

Modern methods include the use of satellite techniques such as the Global Navigation Satellite System GNSS), and satellite altimeters. Satellite techniques can be used to establish and densify three-dimensional networks (Latitude, Longitude and ellipsoidal height) without the need to measure angles and distances between intermediate points more rapidly, with greater accuracy and less difficulty than terrestrial techniques (Poku-Gyamfi, 2009). Survey control could now be established almost anywhere and it was only necessary to have a clear view of the sky so the signal from the GPS satellites could be received clearly.

The Nigerian geodetic control network as shown in Figure 1.1 where green circles represents station points and the yellow lines represents the baselines connecting the stations together forming the control network at the time before the advent of modern space geodetic techniques appeared to meet most user needs were referenced to a non-geocentric datum (based on Clark 1880 ellipsoids) referred to as the Minna to furnish provisional coordinates of stations for mapping purpose as well as to assess the quality of the network. Heights used for the reduction of observations were also obtained.

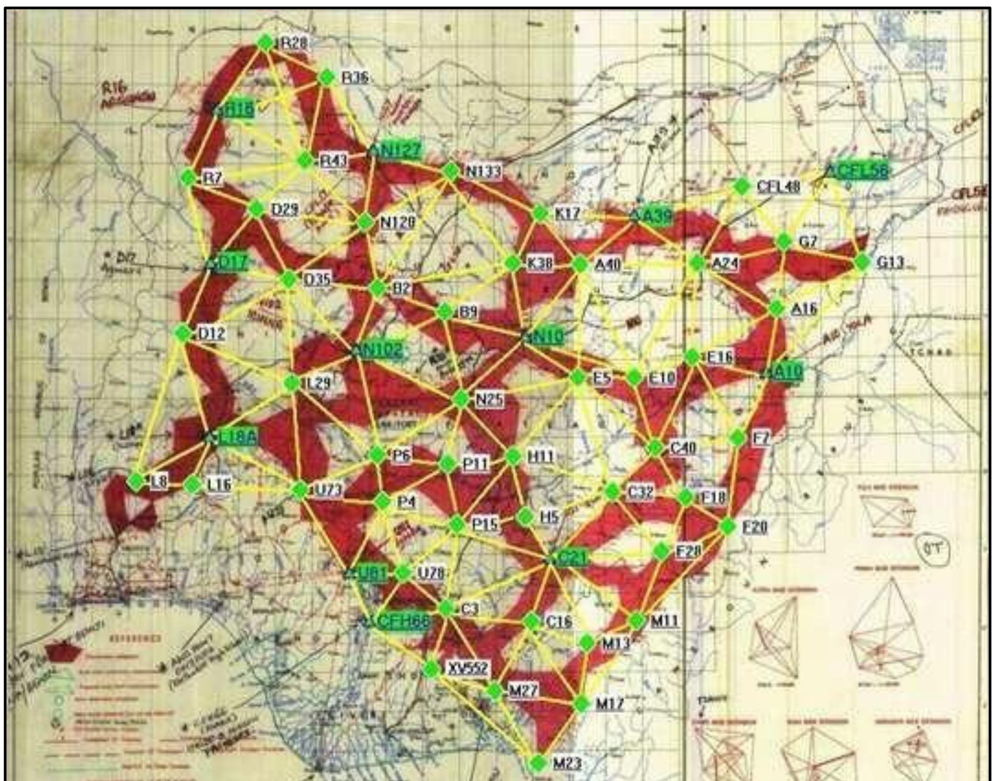


Figure 0.1: Nigeria triangulation network and the primary and secondary traverses. (Nwilo, 2013)

However, the production of the Nigerian primary triangulation network has a number of inherent deficiencies resulting to serious distortion in the network. Some of these problems highlighted by Uzodinma (2005) include:

i. In-accuracy of the scale factor by compression of the Clarke 1880 ellipsoid, thereby causing defect in distances measured.
ii. The origin of the Nigerian network is poorly defined
iii. There is absence of geoidal height model
iv. Difficulties in the determination of the transformation parameters

According to Arinola (2006), most of the stations have neither been visited nor utilized in any manner since their establishment. During the last decades, many geodetic pillars materializing the reference frame have been destroyed, and only a small percentage of beacons are still usable. Also, in the existing framework, controls are very sparse in some areas of the country making it difficult to reference landed properties to the National grid. Another common problem with this network is the obliteration of the reference point monuments, which has rendered most of the points unusable as they are sometimes inaccessible. It is difficult and expensive under the conventional survey methods, to re-establish them after they have been tempered with or destroyed, usually done out of ignorance. Moreover with the distortion of about 10m discovered in part of the network, it cannot meet the requirements of new technologies such as GPS and capable of creating an irreconcilable problem during the implementation of a country wide GIS where data from different sources have to be integrated (Arinola, 2006).

The need strengthen and re-enforce the existing first order Nigeria Geodetic Network caused the geodetic community and the National Mapping Agency of Nigeria; the Office of the Surveyor General of the Federation (OSGoF) to examine the integrity and reliability of the existing geospatial/geodetic infrastructure to meet and satisfy modern and future needs by leveraging and utilizing advancements in GNSS having the adopted WGS 84 ellipsoid as its global datum which is consistent with the International Terrestrial Reference Frame (ITRF) (Dodo, Yakubu, Usifoh, and Bojude, 2011).

Measurements made from these solutions will provide direct compatibility with GNSS measurements and mapping or Geographical Information System (GIS) without the need for unnecessary transformation by spatial users and allow more efficient use of an organization spatial data resource by reducing need for duplication and unnecessary translation and reduce the risk of confusion as GNSS, GIS and navigation systems become more widely used and integrated into business and recreational activities in Nigeria (Isioye and Fajemirokun, 2011). These solutions can also serve as a base for the realization and implementation of the National Spatial Data Infrastructure (NSDI) and other geodetic network applications.

In summary, the conventional geodetic network is not compatible with the current satellite techniques, which does not ensure compatibility across various geographic, land and survey systems at the global level by a common coordinate reference system through which all types of geo-referenced information can be interrelated and exploited reliably (Isioye and Fajemirokun, 2011).

Therefore with the aid of GNSS technology, adjustment of part of the Nigerian primary triangulation network can be achievedfor the purpose of all related survey and other geodetic can serve as a backbone of new precise geodetic networks that makes positioning system easier and accurate enough to meet modern needs.

1.2 Statement of Research Problem

The Nigerian geodetic control network has served the mapping and cadastral needs of Nigeria well over the past sixty years. However, with the discovery of errors within the system which appears to increase away from Minna (triangulation point L40) the origin of Nigerian Survey (Arinola, 2006), inherent problems and limitations of the local system such as inefficiencies and difficulties to relate to modern system hence, the need to embrace a global reference system that is at harmony with all other countries of the World (Nwilo, Dodo, Edozie, and Adebomehin, 2013; Abudu and Adebomehin, 2016).

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). With more satellites being deployed, modernization programs on-going to improve systems against interference, additional signals for better atmospheric modeling, and receiver costs becoming cheaper, satellite positioning techniques provide brighter future for datum definition and control networks densification. These facts coupled with accuracy, speed, adaptability and flexibility in operation make space-based technologies effective tools for the acquisition of geo-information (Nwilo, 2013).

Positioning with certain accuracy implies time transfer capability with comparable accuracy.The high demand for accurate, and reliable positioning using GNSS for many survey applications lead to the advent and wide usage of different positioning techniques. Global Navigation Satellite Systems provides various types of positioning state solutions.Positions can now be determined in different ways depending on what the position is determined with respect to defines the positioning mode.

This competitive positioning approaches have varying significant benefits over each other in terms of operational cost and complexity, efficiencies and different level of accuracy for many spatial applications at various fields.Depending on specific use of data and type of work being performed, there will be different needs for the accuracy of the locational data for different surveying applications thus the need to ascertain the level of accuracy performance different types of GNSS positioning can delivered for proper survey application.

In summary, different GNSS positioning techniques gives rise to different solution quality in post processing mode and these positioning techniques are usually employed without adequate knowledge of the accuracy determination capability of the techniques. Hence, the need to evaluate their positioning performance in terms of accuracy and reliability for suitable application on the ever increasing GPS applications.

The study attempts to answer the following questions;

i. What is the geometric status of the existing primary network in Nigeria?
ii. What are the points solutions based on baseline, network and precise point positioning approaches for the primary geodetic stations?
iii. What are the differences between the results realized by various solutions with reference coordinates?

1.3 Aim and Objectives

The aim of this research study is carry out 3-D readjustment of part of the Nigerian Primary Triangulation Network with GNSS data.

The objectives of the study are to:

i. Determine the geometric status of the existing primary triangulation network in Nigeria.
ii. Generate GNSS point solutions based on baseline, network and precise point positioning approaches for the primary geodetic stations
iii. Comparative evaluation of the various solutions with reference solution coordinates

1.4 Justification of Study

The national geodetic network is a pivotal infrastructure of any country by providing the foundation for all geo-referencing activities. It is the base for coherent multipurpose Land Information System (cadastre) and its subsequent maintenance. Such system plays a vital role in the economic development of the country by delimiting and monitoring changes in property, environment, and biodiversity. The geodetic network services included are not limited to land management, urban development, physical planning, the construction industry, mineral exploration, investment and road construction. It is also vital to both air and water transport. The geodetic network is very important in the management of land in a decentralized system. It is vital in the smooth implementation of the National Land policy and can also help in generating direct revenue to the local governments (Jatua, *et al.*, 2010).

GNSS, an advanced improved method of surveying capable of giving high accuracy and reliable result with a standard error in sub-centimeters has gradually been replacing traditional procedures for conducting precise surveys. Moreover, the use of classical methods is limited by such requirements as inter-visibility between the instrument stations and target stations, favorable weather and atmospheric conditions and accessibility of stations (nature of the terrain). The classical networks established by terrestrial methods are insufficient to contemporary requirements but with GNSS, geodetic measurements that are consistent in three dimensionsover larger distances can be determined with cm-accuracy is possible. It is also easy to use, portable, less labour intensive, and coordinates of the GPS are referenced to the World Geodetic System (WGS 84), a global ellipsoid having its origin as the mass centre of the earth, and height, compatible with international systems.

1.5 Scope of Study

For the purpose of this study, limited available data were obtained from Office of Surveyor General of the Federation (OSGoF) consisting of fifty-six (56) first-order coordinates of stations referenced to Clarke 1880 and fifty-two (52) GNSS observational raw station data along with same points processed earlier with BERNESSE software. The GNSS campaign observations were processed using freely available software at no cost. Three processing strategies were used for the study. Trimble Total Control software was used to process two out of the three processing techniques (Baselines and Network approach) while GNSS-LAB (gLAB) tool was used to process the third approach (PPP technique) for evaluation.

1.6 The Study Area

Nigeria is located in western Africa on the Gulf of Guinea and has a total area of 923,768 km2 (356,669 sq mi) making it the world's 32nd-largest country (after Tanzania). It shares a 4,047km border on the west coast by the Republic of Benin (773 km), on the north by the Republic of Niger (1497 km), on the east by the Republic of Cameroon (1690 km), and on the south by the Atlantic Ocean with a coastline of at least 853 km. (https://en.wikipedia.org/wiki/Nigeria). Nigeria lies between latitudes 4° and 14°N, and longitudes 3°E and 15°E is divided into thirty-six states and a Federal Capital Territory (FCT) as shown in Figure 1.2. Nigeria has a varied landscape; to the southwest of the

Niger is a rugged highland and to the southeast of the Benue is the Mambilla Plateau, which forms the highest Plateau in the country. The highest elevation point in Nigeria (2,419metres above sea level) is located in ChappalWaddi in the Northern state of Taraba (https://en.wikipedia.org/wiki/Nigeria).

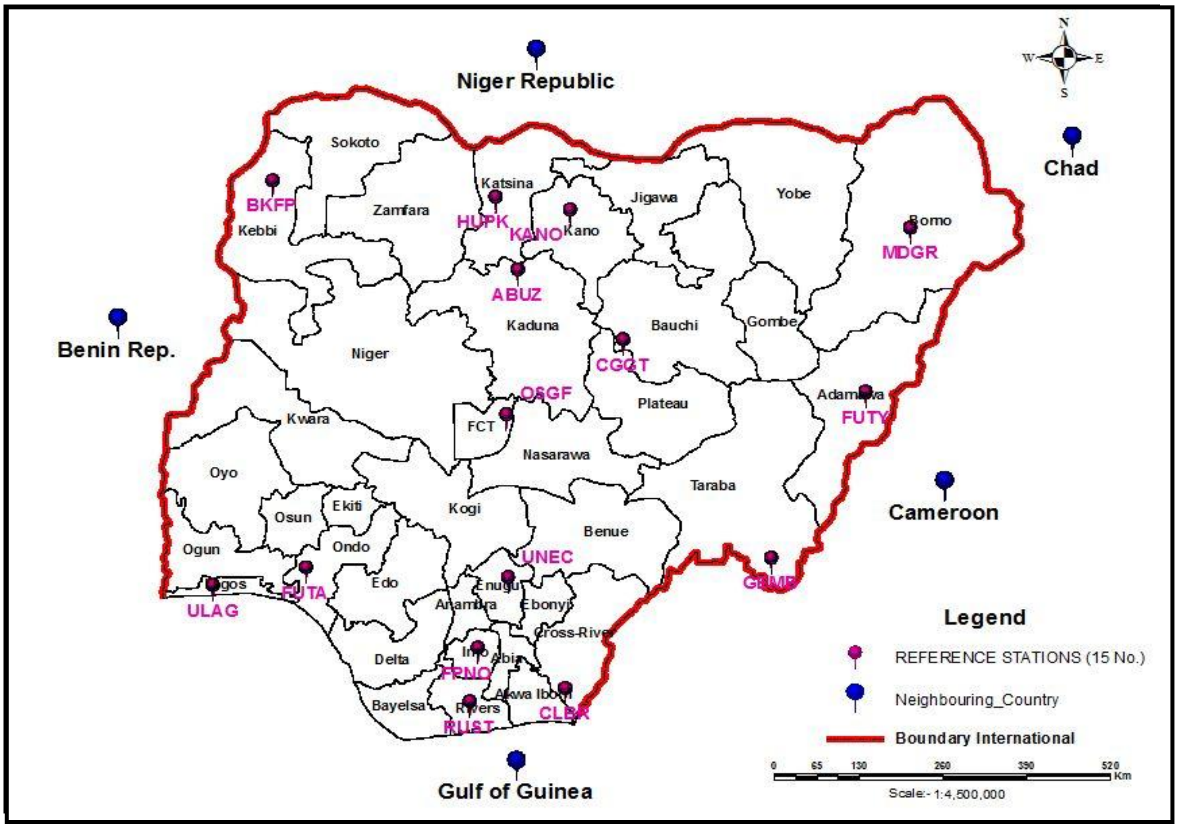


Figure 0.2**:** Map of Nigeria with the Nigerian Permanent GNSS Reference Network (NIGNET). (Source: OSGoF)

CHAPTER TWO

THEORECTICAL FRAMEWORK AND LITERATURE REVIEW

2.1 Geodetic Reference Frame

In geodesy, a reference coordinate frame implies a scale, orientation and coordinate origin as part of a reference system and the changes in these parameters over time. The origin of a reference frame is the zero point of the three Cartesian axes (i.e., X, Y, and Z), typically the center of mass of the entire Earth system. This point can be determined most accurately from the observations of satellite motion, as satellites naturally orbit about Earth's center of mass. The "scale" refers to the absolute distance between points in the network. It is a uniform scaling of all coordinates, with the result that a scale error in the reference frame results in a radial (height) error for all stations. The "orientation" essentially refers to the definition of the zero point for longitude and latitude while a reference system is a mathematical abstraction; its practical realization through geodetic observations is known as a reference frame, (Kouba and Popelar, 2009). These reference systems are the foundation for all operational geodetic applications associated with mapping and charting, navigation, spatial data acquisition and management, as well as support for the geosciences. There are three basic types of geodetic reference frame which are briefly explained in the following section:

2.1.1 Global reference frames

A global reference frame is typically the primary basis for the definition of a coordinate system.Examples include the International Terrestrial Reference Frame (ITRF) and the World Geodetic System 1984 (WGS 84). These frames are geocentric in nature, having the geo-center (the center of mass of the Earth) as the origin and orthogonal axes aligned with pole, equator and Greenwich meridian according to IERS conventions (Acheampong, 2008).

2.1.2 World Geodetic System (WGS84)

WGS84 is a global geodetic reference system which has been established and maintained by the U.S. Department of Defense to facilitate positioning and navigation worldwide to provide a basis for expression of inter- continental geodetic information (DMA, 1991). The terrestrial coordinate reference frame corresponding to WGS84 has been updated to keep pace with increasing precision of GPS positioning and navigation technology in general use. The reference frame is very similar to ITRF in specification. The latest realization was released on February 8, 2012 and is known as WGS 84 (G1674). The defining parameters of the WGS 84 ellipsoid include :(Uzodinma, Oguntuase, and Dimgba, 2013).

Semi-major axis (a) = 6378137 m, and Flattening (f) = $1/_{298.257223563}$

2.1.3 International Terrestrial Reference Frame (ITRF)

The ITRF is a global, geocentric 3D reference frame co-rotating with the Earth's crust in its diurnal motion in space. The most recent and refined realization of the ITRF is the ITRF 2008, with the reference epoch 2005.00 and full variance-covariance information and could be used to express position of points. ITRF has been implemented and maintained to satisfy the highest accuracy positioning requirements on the global scale. The ITRF is realized by the coordinates and site velocities of a network of global stations and forms the basis for modern regional and national reference frames or geodetic datum. The fundamental techniques through which these measurements have been acquired include GNSS satellites (GPS, Global Orbiting Navigation Satellite System (GLONASS)), SLR, VLBI, and DORIS. (http://itrf.ensg.igs.fr).

Each of the observational techniques has unique characteristics. VLBI connects the ITRF to the celestial reference frame and is important for realizing the scale accurately. SLR is the satellite technique that is used to locate the center of mass of the Earth system, and so defines the origin. GPS primarily contributes to the number of sites that define ITRF (densification of ITRF), and to monitoring polar motion precisely. GPS, DORIS, and SLR are used to position Earth orbiting satellites in ITRF, and GPS is used to position points and their velocities on the Earth' s land and sea surfaces, such as benchmarks, tide gauges, and buoys. DORIS is the geodetic technique with the most homogenous station distribution, implementation, and operation (Fagard, 2006). Connections between the techniques are enabled by collocation at a subset of ITRF sites where two or more space geodesy instruments are operated and local - site ties between monuments are measured using terrestrial high - precision surveying techniques. Conventional precise surveying techniques have been used for decades to connect different techniques, and precise leveling is still a critical method for establishing the vertical tie between tide gauges and local benchmarks.

Significantly, this reference frame can be extended to all regional and local studies in order to link multidisciplinary observations and ensure long - term consistency, precision, and accuracy. ITRF solutions can be specified in by Cartesian equatorial coordinates X, Y, Z and also transformable to geographical coordinates (ϕ, λ, h). Consequently, the most rigorous way for geodesists to achieve centimeter level accuracy is to perform GPS measurements relative to ITRF control stations (Dodo *et al*., 2011).

The International Terrestrial Reference Frame (ITRF) is the most accurate reference frame that exists internationally and consequently more countries are using national solution based on ITRF. Adopting an ITRF based geodetic datum allows for a single standard for collecting, storing and using geographic or survey related data. This has ensured compatibility across various geographic, land and survey systems at the local, regional, national and global level by a common coordinate reference system through which all types of geo-referenced information can be interrelated and exploited reliably (Isioye and Fajemirokun, 2011).

2.1.4 Regional reference frames

Regional reference frames are denser networks of geodetic stations covering continental areas. As with ITRF, regional reference frames are defined by the coordinates and site velocities of contributing stations Examples include the European Terrestrial Reference Frame (EUREF), North American Datum 1983 (NAD83), African Reference Frame (AFREF), Sistema de ReferenciaGeocentrico para las America (SIRGAS) and the Asia-Pacific Reference Frame (APREF). Regional frames not constrained by the motion of a single tectonic plate are closely aligned with ITRF.

2.1.4.1 African Geodetic Reference

Since the satellite technology has been developed and widely used, the reference system is trended to the world reference system which basically leads to one common system and frame. All geo-information based on one common system could support to predict and solve many global scientific problems such as climate, environmental, and natural disasters. There are more than fifty (50) countries in Africa with different reference system and datum. To keep pace with international trends, Africa's attempt to harness this technology has made it imperative to unify Africa's geodetic reference frame. It is also an effort to make co-ordinate planning and development within and across countries easier (Wonnacott, 2005).

The limited success in the previous attempt to unify Africa's datum through the African Doppler Survey (ADOS) was not a discouragement in establishing AFREF whose objectives according to Kufoniyi (2011) were to:

i. Define and establish a continental geodetic reference frame for Africa through a network of permanent GNSS stations such that any user anywhere in Africa would have free access to GNSS data and products at most 1000km from such stations
ii. Establish precise and uniform African Geoid
iii. Determine transformation parameters between GNSS and ITRF to/from local reference systems
iv. Promote the use and application of GNSS technology for Africa's development
v. Identify the necessary geodetic requirements of participating nations and international agencies etc.

However, in order to effectively implement these objectives and to avoid the "ADOS Failure" an intermediate structure was proposed at the sub-regional level to enhance coordination, which resulted in sub-regional reference frames: NAFREF (for North Africa), SAFREF (for Southern Africa), CAFREF (for Central Africa), EAFREF (for East Africa) and WAFREF (for West Africa), all still conforming and compatible with IGS/ITRF specifications. Following the principle of national implementation, countries will be expected to maintain and secure the stations, undertake field campaigns and submit the data to designated regional data centres. It is noted that countries may not be fully self-sufficient in terms of the resources required to establish and maintain such a station. Also some countries may have more responsibilities than others. Assistance may therefore be sought for such countries from other African countries that have more capacity and from the international community (UNESC, 2003).

2.1.5 National reference frames

Modern national reference frames are typically a static realization of ITRF or a regional reference frame and the coordinates are considered to be invariant with time. Traditionally each country has its own geodetic reference system resulting in non-compatible coordinates systems between countries. Most of them were confined to small areas of the globe, fit to limited areas to satisfy national mapping requirements. Maps in neighboring countries do not match at the national boundaries (Jatua*et al*., 2010; Abudu and Adebomehim, 2016).

In most countries the coordinates of a national reference frames (or geodetic datum) form the basis for all surveying, positioning and mapping within national borders because it best approximates the size and shape of that particular part of the Earth's sea-level surface. It is defined by specifying a reference ellipsoid, the position (latitude and longitude) of an initial station and an azimuth from that station. Invariably, the centre of its ellipsoid will not coincide with the Earth's centre of mass. The mapping systems of various countries are based on their local coordinate systems. In the local coordinate systems, horizontal positions are referenced to the local ellipsoids that are defined differently by various countries to fit their topography, and heights are referenced to the local geoid (orthometric height, H) (Uzodimma*et al.*, 2013).

2.2 Global Navigation Satellite System (GNSS) Measurement

The advent of GNSS has solved many of man's challenges in terms of accurate positioning as against the traditional method of positioning using the celestial bodies with analogue instruments and techniques. Today, with the application of GNSS the geographical geocentric position of any point on any part of the earth can be determined by carrying out observations on the earth-orbiting GNSS satellite with ease and speed. GPS has been used for high precision geodetic survey, engineering and topographical surveys (via post processing and real time techniques). Two GNSS systems are currently in operation: the United States' Global Positioning System (GPS) and the Russian Federation's Global Orbiting Navigation Satellite System (GLONASS). A third, Europe's Galileo, is slated to reach full operational capacity in 2020. Each of the GNSS systems employs a constellation of orbiting satellites working in conjunction with a network of ground stations. (Uzodinma, Oguntuase, and Dimgba, 2013).

Satellite-based navigation systems use a version of triangulation to locate the user, through calculations involving information from a number of satellites. Each satellite transmits coded signals at precise intervals. The receiver converts signal information into position, velocity, and time estimates. Using this information, any receiver on or near the earth's surface can calculate the exact position of the transmitting satellite and the distance (from the transmission time delay) between it and the receiver.

2.2.1 GNSS observables and algorithm

There are three fundamental observables for positioning with GNSS namely, pseudo-range (code) measurement, and carrier-phase measurement, subject to errors, systematic and random. The geometric range between the satellite and the receiver is usually distorted by lack of synchronization, the media of propagation, and others.

1. Pseudorange measurement
2. Carrier-phase measurement

2.2.2 Pseudorange measurement

Pseudorange is a measure of the difference in time between signal transmission and reception. Every satellite sends signals at certain time, while the signals get to the receiver later. The measured range is always longer than the geometric distance measurement due to unavoidable errors including the timing, atmospheric errors and orbit errors. Literature on this has been widely written and can be obtained from (Seeber, 2003; Leick, 2005; Dodo, 2009; Ojigi, Eyo, and Bayrak, 2014).

Coordinating current signal data from four or more satellites enables the receiver to determine its position. However, at a given measurement epoch, the GPS receiver generates n pseudorange measurements (from different satellites).

Expressed in general form is given by equation (2.1) (Dodo, 2009):

$$p_{r\,j}^{s} = \sqrt{(x_j - x_u)^2 + (y_j - y_u)^2 + (z_j - z_u)^2 + c\delta t_u} + c\delta t_u \tag{2.1}$$

$p^s_{r\,j}$ = pseudorange measurement from satellite j (m)

x_j, y_j, z_j = ECEF position of satellite j (km)

x_u, y_u, z_u = ECEF position of user u (km)

c = speed of light

δt_u = Receiver clock error (sec)

The pseudo-range observation equation corrected for time is written as equation (2.2)

$$PR^s_r = p^s_r + c[\delta\tau_r - \delta t^s] \tag{2.2}$$

Where,

PR^s_r Pseudo-range observed at receiver r in the time frame of receiver r

p^s_r Geometric range obtained from the true GPS time of the signal left satellite s and the true GPS time the signal arrived at receiver r

$\delta\tau_r$ The receiver clock offset for receiver r in the receiver time frame of the observer

δt^s Satellite clock offset for the satellite s in the satellite time frame of satellite s

C speed of light

2.2.3 Carrier-phase

Geodetic control stations can now be established in the area of interest by relative positioning (differential) using carrier phase observables from GPS satellites. The signal transmitted by GPS satellites consists of two carrier waves namely; L1 and L2. The L1 has a frequency of 1575.42 MHz and wavelength of 19 cm while the L2 carrier wave has a frequency of 1227.60 MHz and a wavelength of 24 cm.

The carrier phase is obtained by adding the true geometric range, speed of light, the integer phase ambiguity, signal wavelength, initial phase the receiver oscillator and the effect of satellite ephemeris errors, satellite orbital error, tropospheric, ionospheric, multipath, and other receiver dependent error for a signal transmitted from a satellite to a receiver. The deterministic model for the carrier phase measurement in meters is given by equation (2.3) (Ojigi, 2014)

$$\emptyset_{(li)} = R^t + c\Delta t_{rs} + d_{trop} - d_{ion/li} + d_{mult/\emptyset(li)} + \lambda_i N_i + \lambda_i \Delta\emptyset_{rs} + \varepsilon_{\emptyset(li)} \tag{2.3}$$

Where,

(Li) is the measured carrier phase on *Li* (m);

*Rt*is the true geometric range (m);

*c*is the speed of light (m/s);

*dtrop*is the tropospheric delay (m);

*dion/Li*is the ionospheric delay on Li(m);

*λi*is the wavelength on Li(m); *Ni* is the integer phase ambiguity on Li (cycle);

dmult/Ø(Li) is the multipath effect in the measured carrier phase on Li (m) and **ε** is the receiver dependent errors known as carrier phase measurement noise (m).

2.3 GNSS Error Sources and Mitigations

Global Navigation Satellite Systems have been playing increasing roles in surveying, geodesy, navigation and other position/location sensitive disciplines. However, these spaceborne systems' accuracy, availability and reliability are subjected to numerous biases or errors. These biases and errors which are determined by the sum of several sources of error reduces the accuracy of GPS measurements that will be discussed in the preceding section and how they can either be prevented or corrected using proven models.

To overcome, reduce or correct for these errors/biases, various strategies, applying correction models and positioning methods must be employed. Some of the models that will be applied to the acquired GPS data in this study to obtain the desired specified accuracy will be discussed in the preceding section. According to Acheampong**, (**2008) the errors are generally in form of;

i. Satellite-Dependent Errors
ii. Receiver-Dependent Errors
iii. Signal Propagation Errors

2.3.1 Satellite-dependent errors

These errors are basically defined in terms of satellite transmitted signals having transmission time from satellite to receiver. It includes error related to satellite clock, satellite geometry, orbit and ephemeris errors.

2.3.2 Receiver-dependent errors

These errors are basically defined in terms of the receivers; it originates from receivers. It includes errors related to clock, cycle slips, receiver noise and antenna phase variation. The offset between the receiver clock time and GNSS system time is the receiverclockerrorthat contaminates all satellite-receiver ranges made at that instant by that receiver. GPS receiver clock errors can be modeled in a manner similar to GPS satellite clock errors. In addition to modeling the receiver clock errors and in an effort to remove them, an additional satellite can be observed during operation to simply solve for an extra clock offset parameter along with the required coordinate parameters.

This procedure is based on the assumption that the clock bias is independent at each measurement epoch. Many of the clock terms cancel when the position equations are formed from the observations during a differential survey session. For receiver clock error, the clock bias should be treated as an additional parameter in the pseudo-range navigation model (Leick, 2005). The International GPS Service (IGS) generates precise ephemerides for the satellites together with by-products such as Earth orientation parameters (EOP) and GPS clock corrections. The GPS orbital information is normally given in the standard SP3 format.

According to Zhang and Lachapelle (2001), the noise for the code measurements can be calculated from the double difference pseudorange using the following equation:

$$\sigma\varepsilon P = 0.5\Delta\nabla P_{rx} \tag{2.4}$$

Where,

σεP Is the measurement noise to account for double difference pseudorange, and

$\sigma\nabla\Delta Prx$ Is the noise variance

The receiver noise for C/A-code measurements is in the range 30-300 cm and for P-code, the values are 3-30 cm. For carrier phase L1, the phase noise would be 0.5-3 mm in a survey-grade receiver. The noise level of the code and phase measurements decreases as the elevation angle increases up to about 45^o and the noise level become constant above 45^{0}elevation. The GPS receiver dependent errors include:

2.3.2.1 Tropospheric Refraction Delay

This delay is a function of elevation and the altitude of the receiver, and is dependent on the atmospheric pressure, temperature, and water vapor pressure. The bias ranges from approximately 2m for signals at the zenith to about 20m for signals at an elevation angle of 10 degrees (Brunner and Welsch, 1993). When GNSS signals which are electromagnetic propagate through this medium, dispersion occurs changing the velocity of the propagated signal. Tropospheric effect can be accounted for by avoiding low elevation satellites and treating the residual bias as a parameter in the final position solution (Roberts and Rizos, 2001)

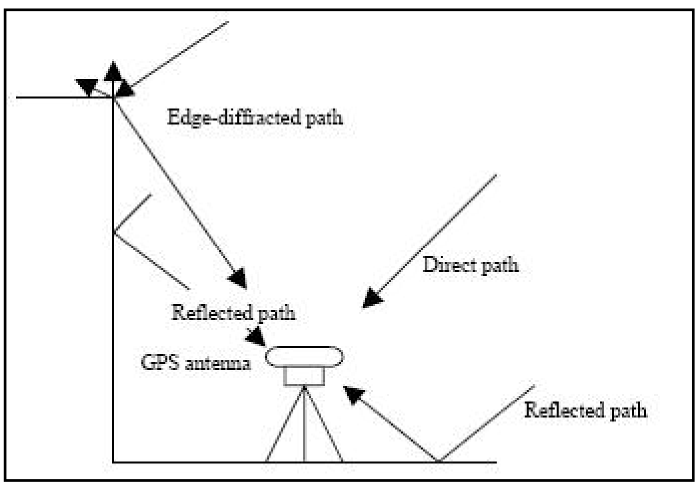
2.3.2.2 The Ionospheric Propagation delay

Dispersion and refraction of the GPS signal is referred to as an ionospheric range effect because dispersion and refraction of the signal result in an error in the GPS range value. The ionosphere and troposphere both refract the GPS signals. This causes the speed of the GPS signal in the ionosphere and troposphere to be different from the speed of the GPS signal in space. Therefore, the distance calculated from "Signal Speed x Time" will be different for the portion of the GPS signal path that passes through the ionosphere and troposphere and for the portion that passes through space. Different models such as the Hopfield model (1969), Saastamoinen model (1973), and modified Hopfield model (Goad and Goodman, 1974) are used to estimate the tropospheric error (Hoffmann *et al.,* 2001).

Saastamoinen Troposphere model will be used to estimate and mitigate the effect of tropospheric error.

2.3.2.3 Multipath

Multipath as depicted in Figure 2.1 describes an error affecting positioning that occurs when the signal arrives at the receiver from more than one path. Multipath normally occurs near large reflective surfaces, such as a metal building or structure. GPS signals received as a result of multipath give inaccurate GPS positions when processed. However, with the newer receiver and antenna designs and sound prior mission planning to eliminate possible causes of multipath, the effects of multipath as an error source can be minimized. Averaging of GPS signals over a period of time can also reduce the effects of multipath (Ogaja, 2002).



. Figure 0.3 : Multipath (Ogaja, 2002)

2.4 Precise Point Positioning (PPP) Correction Models

PPP has become a popular technique for processing GNSS data. The Precise Point Positioning observation model includes a number of effects affecting the GNSS satellites, the propagation of the signal and the station receiver such as relativity, satellite and receiver antenna phase center variations, phase wind-up biases and station displacement.

2.4.1 Tidal deformations

Due to tidal deformations that occurs in the solid earth because of certain degree of elasticity, solid earth tide is accurately computable. The elastic earth responds to several external forces causing site displacement. Theresulting effects can reach up to several decimeters and have to be modeled in order toachieve the ultimate accuracies. The Earth tides are similar to ocean tides. In order to achieve truly centimeter-level global geodesy, solid tide loading must be included in the site positional analysis (Witchayangkoon, 2000; Karabatic 2011).

2.4.2 Phase wind-up error

Correcting the phase wind-up error is expected to improve mostly the receiver clock estimates. Phase wind-up problem is associated with the antenna orientation, both at the satellite and at the receiver. This is due to the electromagnetic nature of circularly polarized waves intrinsic in the GPS signals. Ideally, at the receiver the measured angle of carrier phase equals the geometric angle between the instantaneous electric field and a reference direction at the receiving antenna. Thus, when the antenna orientation changes, so does the reference direction, and subsequently the measured phase. Likewise, the change of satellite antenna orientation changes the direction of the electric field at the transmitting antenna and, as a result, the change at the receiving antenna, thus the measured phase (Witchayangkoon, 2000; Karabatic 2011).

2.4.3 Relativistic effects

The relativity correction is important for Precise Point Position (PPP). If it is not applied, the PPP solution accuracy will degrade to several meters**.** The synchronization of stable accurate clocks on board the satellites and the ground receivers are of high importance. However, the time transfer between the satellite and ground receiver is affected by relativistic effects.

The GPS receiver time (t) (after applying the clock corrections) observed at the ground is obtained as:

$$t = t^s - \Delta t_{rel} \qquad (2.5)$$

where t^s is the time on board the satellites either calculated from the coefficients included in the broadcast message, or given in the IGS clock solution files to be applied (Kouba, 2004).

2.4.4 Satellite antenna phase center offset and variation

The signal transmission and phase measurement are related to the satellite antenna phase center causing an offset. The antenna phase center offset (PCO) for each satellite antenna type and variable offset relative to the mean phase center variation; phase center variation has to be known as given in eqn (2.6) it is usually accounted for in the calculation using a publicly available satellite information read from ANTEX formatted file available at IGS server (Witchayangkoon, 2000; Karabatic 2011).

$$X_{scm} = x_{apc} + PCO + PCV \qquad (2.6)$$

Where,

X_{scm} is the position of the satellite center of mass

x_{apc} is the satellite's antenna phase center vector

2.4.5 Receiver antenna phase center

The receiver antenna phase center offset can cause errors in the up coordinate by as much as 10 cm and sub-centimeter errors in the horizontal and needs to be corrected for. The offsets from the receiver's antenna reference marker and the receiver's antenna phase center are spaced and dependent. Also, the correction for the antenna phase center variation accounts for a signal direction and the frequency dependence. In addition, radomes that are used for the GNSS receiver antennas influences the antenna phase center variations. These corrections are also generally obtained through a calibration and are listed in a publicly available file (Witchayangkoon, 2000; Karabatic 2011).

2.5 GNSS Network Positional Concept

2.5.1 Single network base

Single Baseline Network positioning system gives a level correction obtained by using either a personal base station or a single CORS base station. Basically, a single baseline solution uses one base station to obtain correction data. This single base station can be a personal base station setup or a CORS station etc. and is accessible either by radio transmission or cellular modem via the internet. For radio transmitted data, it is recommended that the rover unit be within 6(mi) or line-of-sight to ensure adequate signal integrity. For data accessed via the internet, it is recommended that the rover be within a 20-25(mi) radius of the base station being accessed. The distance in this scenario is a function of satellite commonality. In other words, the rover and the base station must be able to see the same constellation of satellites (minimum of 5) for accurate correction data and to achieve position quality. Precise observations using dual-frequency carrier phase can be obtained using single base line network processed using a differential GNSS (DGNSS) algorithm, such as real time kinematic (RTK) or post-processing (PP). The process is on assumption that the quality of the reference station data is consistent with the desired level of positioning accuracy. (www.AlabamaPrecisionAgOnline.com).

2.5.2 Precise Point Positioning (PPP)

PPP is a technique that uses un-differenced single- or dual-frequency pseudorange and carrier phase observations of a single receiver along with accurate orbital data, accurate satellite clock data, and uses the ionosphere-free function of the observables to generate high accuracy positioning solutions. It eliminates the need for simultaneous observations at both the reference and rover receivers; it also eliminates the needs for the rover receiver to operate within the vicinity of the reference receiver; and it provides homogenous positioning quality within a consistent global frame anywhere in the world with a single GNSS receiver. Precise point positioning technique relies on the signal transmitted from one or multiple global navigation satellite systems. Omitting the building of baselines significantly reduces the required processing time compared to the network (baseline) solution (Witchayangkoon, 2000; Karabatić, 2011; Guo *et al.*, 2016)

2.5.3 Multiple reference network

Multiple reference Network is typically made up of multiple base stations that work concurrently to provide correction. Typically, five or more base stations are needed to derive a network solution, but as few as three may be used in some instances. If a rover happens to be located within a cluster of base stations but outside of the 20-25 mile recommended radius of any one station. In addition to offering accurate correction outside of the single baseline recommended distance, multi reference network provides increased reliability over a single baseline solution. If one base station goes down in the cluster, another station can typically take its place without interruption from the rover's perspective. The rover thinks it is operating in the immediate vicinity of a real base station, but in fact, service is modeling error experienced at the location and broadcasting it over. Basically, a master station is determined, either by the user or automatically, which is typically the closest base station to the rover (www.AlabamaPrecisionAgOnline.com).

Data from multiple stations are used to derive a network correction at the server then broadcasted to the rover. In the classic method using a single reference station, the rover needs to work within a distance of 1-15 km from the reference station, and no redundancy of the reference stations is usually available (Petovello, 2011). In the network approach of multi-reference stations, such redundancy exists, and the user no longer needs to establish his or her reference station. The distance to the closest reference station can be extended to several tens of kilometres, network solution works on reducing bias sources, particularly distance dependent errors, to produce a fix. The network approach usually requires a minimum of three reference stations to generate network corrections. As the number of stations increases, redundancy increases, and better corrections can be estimated. (www.AlabamaPrecisionAgOnline.com).

This approach allows a more precise modelling of distance-dependent systematic errors principally caused by ionospheric and tropospheric refractions, and satellite orbit errors. More specifically, a GNSS network decreases the dependence of the error budget on the distance of nearest antenna. Using this technique, highly improved positioning can be performed inside the network area. The precision of positioning reaches two centimetres in the horizontal plane and four centimetres in the vertical direction (2σ). Another benefit multi reference data are free of site-specific errors such as multipath, because the computation assumes that the virtual station is situated at an ideal location.

In providing a network solution, different approaches are usually employed to obtain positional fix of the unknown as found in (Wells, N., …, 1999).

i. Reduce the observation campaign to simultaneous data observed from pairs ofstations (baselines), and then combine these baselines into a network.
ii. Take the GPS observations observed simultaneously at all stations directly into a network adjustment where all the coordinates of the network are present as unknowns.

The first approach is rigorous if there are no common observations and biases between baselines. With an array of interconnected baselines, considering one baseline as the basic unit, then the basic model for a three-dimensional adjustment is the following:

$$\Delta R_{ij} = R_j - R_i \quad C_{\Delta R} \tag{2.7}$$

Where,

ΔR_{ij} is the quasi-observed value of the coordinate difference

$C_{\Delta R} =$ Covariance Matrix

2.6 Review of Related Works

Many countries have successfully modernized their geodetic datum to a geocentric realization of ITRF at different epoch as the basis for their national datum. Examples of such countries include South Korea KGD2002 ITRF2000 at 2002.0 reference epoch, Japan JGD2000 ITRF2000 at 2000.0 reference epoch, New Zealand NZGD2000 ITRF96 at

2000.0 reference epoch, Indonesia DGN1995 ITRF2005 at 1995.0 reference epoch, Australia GDA94 ITRF92 at 1994.0 reference epoch, China CTRF2000 at ITRF97 2000.0 reference epoch (Dodo *et al.*, 2011).

Mtamakaya (2009), carried out a study that successfully provided the basis for the establishment and maintenance of a 3-dimensional dynamic reference frame for Tanzania in the ITRF system with optimum accuracy requirement in an economic way by developing a conceptual plan to realize a new spatial framework in Tanzania (TZRF10) using the least-squares covariance analysis technique as a tool to evaluate for its geometrical consistency using the BerneseV5.0 scientific software.

Poku-Gyamfi (2009) established a Geodetic Reference Network (GRN) for Ghana covering the Golden Triangle that comprised three (3) permanently operating reference stations which serve as primary control points and seventeen (17) second order network point located in and around the Golden Triangle of Ghana. The reference network was based on ITRF05 and realized using PrePos GNSS Suite Software. However, with a new corridor concept introduced to the network for atmospheric correction was found to be reliable but with a lot limiting factors affecting the accuracy of the data.

Bundoo (2013) established a reliable first order geodetic reference network in Monsterrado country–Liberia using GNSS technology. The newly established reference network consists of six (6) primary points, covering an area of approximately 11.693Km2 with an average separation of about 8Km. The new network monuments was 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.

Moka (1987) carried a study on the Evaluation of the D-Chain of the Nigerian Triangulation Network adjustment of the Nigerian triangulation network. Various approaches which include phase and sequential adjustment were used for the final adjustment. A computer program based on single vector form of the variable band width normal equations matrix was used to analyze portions of the Nigerian horizontal geodetic network, to determine the quality of observation and to identify areas of weakness. The network was tested using least squares adjustment of Nigeriantriangulation network and a posterior standard error of an observed angle of unit weight. The analyses of the residuals of the angles showed a normal distribution of the residuals and there were no blunders in the observations. The computed average standard errors of adjusted azimuth and distance of some selected lines, which were criteria for judging the strength of a network, fell within the specification for first order horizontal controls and that the Federal Surveys coordinates were close to the most probable values with the triangular misclosure not exceeding 1.2mm and 2 – 5mm respectively.

Arinola (2006)determined the magnitude of error in the Nigeria control network using Single frequency post processing (LI), GPS Promark II and Dual frequency (LI, L2) GPS (Radian IS by Sokkia) with RTK capability to deployed to carry out GPS observations in static mode with a minimum observational time of four (4) hours. The result revealed a positional error of the magnitude of 9m in the network.

Alnaggar and Dawod (1995) in an attempt to increase the reliability of GPS geodetic networks developed a special GPS adjustment program with a built-in automatic outlier detector which utilizes the τ (TAU) statistical test using a 20-station GPS network to examine and investigate the quality of outlier detection. Significant improvements were obtained after identifying and removing erroneous observations

Fortes *et al* (2001) carried out a study on improving a Multi-Reference GPS Station Network Method for OTF (On-The-Fly) Positioning to provide cm-level positioning accuracies, as long as the carrier phase ambiguities are resolved on-the-fly (OTF) to integer values. To them this new approach was developed by using least square collocation to separately model the errors into tropospheric, satellite orbital components and .modelling the ionosphere into directional components, which has shown to be relevant under high ionospheric conditions. The results of the outcome presented showed enhanced method using data collected in the St. Lawrence Seaway region, Canada, and compared with the ones obtained modelling the total error in L1 and Wide Lane carrier phases.

Different types of software have been employed for processing, adjustment, computation and subsequent analysis of global, regional or national GNSS geodetic data that requires high accuracy. Among these include Bernesse, GAMIT, Gipsy, FillNET, TurboNet, etc for example, Popovas and Radzeviciute (2001) carried out a study on readjustment of the Lithunian GPS Network using GAMIT processing software. The previous adjustment done in 1993 and the technical base for this task were done combining several software packages such as GPS (for baseline processing), FillNet and TurboNet (for network adjustment). These adjustment programs had some restrictions mainly because of hardware limitations and were time consuming. The mentioned programs were based on processing separate sessions and such steps as correlation between different sessions were neglected. However, with GAMIT/GLOBK software full adjustment was done. The achieved results are better than results from previous adjustment.

Ahmed (2003) carried out a study on design parameters of multi-reference station GPS networks impact on positioning performance and phase ambiguity resolution was analyzed. The main parameters considered were the distances separating reference stations, network configuration, communication between the computing centre and the user, and network algorithm. A number of tests have been conducted in a case study utilizing the Dubai Virtual Reference Network System (DVRS) in the United Arab Emirates; different network configuration was also examined, consisting of variable combinations of the DVRS and the auxiliary stations. In addition, positioning results of two network algorithms (FKP/VRS and the Multi-reference methods) were compared. The test result shows that a major technical challenge in the design of multi-station networks is the large distances between reference stations, which has a direct impact on positioning accuracy and initialization of the carrier-phase measurement ambiguities. To achieve a fast and reliable integer ambiguity resolution, it is recommended to select average baseline lengths of 50-70 km; with a maximum value of 100 km. increased redundancy of reference stations improves positioning accuracy and ambiguity resolution.

Robert (2007) carried a study on Network triangulation; a method for estimating distances between nodes in the network, by allowing every node measure its distance to a few beacon nodes, and deducing the distance between every two nodes *x; y* using the measurements to its respective common beacons and applying the triangle inequality.

The outcome indicates that known constructions may potentially be improved (such as redefining the triangulation and/or its analysis, or extending the triangulation. Also, certain quantitative trade between accuracy and slack are unavoidable. A possible workaround is to relax the methodology and obtain only an upper estimate or only a lower estimate; such an estimate may provably give a good approximation to the true distance

Abdallah and Elsayed (2007) developed a simple model for improving the accuracy of the Egyptian Geodetic Triangulation Network using available precise coordinates at some stations of the network in defining the distortion in the network and to establish a correction model to improve the accuracy of the local network without readjusting the network. It was discovered that the proposed model was much closer to the accurate reference than the rigorous adjustment at the available stations. However, even though the results were obtained through the available common station, more points with high accurate WGS- 84 coordinates are needed to assure those results and to make them more realistic

Silvia and Juraj (2010) worked on Adjustment of positional geodetic networks by unconventional estimations, the technique used in the study is based on the unconventional estimations of repeated least-squares method with gradual changing of weight of individual measurements until no deviated measurements were found in the file of the measurement. MatLab programme version 5.2 was used to implement the mathematical adjustment. This method has several advantages – in case of normality of the sets of geodetic data measured it leads to the most reliable estimates of unknowns and provides statistically unbiased and consistent estimates of parameters. The techniques allow obtaining estimates with various necessary properties and avoiding hidden, serious and systematic errors in measurements which could essentially effect and the value of parameters being fixed. The method of least squares was used for mutual adjustment of the satellite and terrestrial measurements

Result of their mathematical adjustment showed conformity of the individual tested robust estimation methods. Based on the results obtained in the processing of their experimental geodetic network it's reasonable to suggest that the adjustment methods used not only produced comparable results but also shows similar graphic course of the functions of individual estimation procedures.

Nicolae*et al.* (2014) carried out a research on**the possibilities of the geodetic support network development by using the national triangulation network** in *Romania.* The result of the study showed final accuracy of a geodesic support network positioning is given both by the absolute accuracy obtained from the processing and compensation into the WGS84 reference system, and by the accuracy of the coordinate transformation to the national reference system. Internal geometry of the network has a significant effect in achieving the requirements of accuracy, and uniformity. It specifically counts the network's geometry and placement coordinate transformation.

Numerous researches have shown millimeter-to centimeter-level point positioning accuracy can be achieved for static dual-frequency PPP processing using a 24-hour good quality dataset (Colombo *et al*., 2004; Gao and Shen, 2005; Heroux and Kouba, 2001; Kouba, 2009; Witchayangkoon, 2000; Zumberge*et al*., 2001)

Henriken et al. (1996) tested stand-alone positioning with single- and dual- frequency pseudoranges. Using precise ephemerides and clock corrections, the post-processed analysis was conducted based on epoch-to-epoch solutions. The low passed filtered Fast

Fourier Transform (FFT) was then applied to remove high frequency receiver noise. The results are accurate to 0.5-1.5 m horizontally and 1.5-3 m vertically depending upon how one accounts for ionospheric corrections.

Witchayangkoon (2000) investigated Precise Point Positioning (PPP) using dual and single-frequency pseudorange and carrier phase observations in static and kinematic modes. The static PPP solution examples used six-hour data sets from four stations. The observations were made while Selective Availability (SA) was active and after it had been discontinued. The static solutions agree to within 10 cm with published coordinates. The kinematic solutions showed a discrepancy of less than one meter, mostly around half a meter. For observations with low multipath, the research shows that single-frequency ionosphere-free PPP solutions are equivalent to the dual-frequency solutions. All solutions incorporated corrections for solid earth tides, relativity, and satellite antenna phase center offsets.

Karabatic (2011) showed how the atmospheric precipitable water content derived from GNSS data can be assimilated within an operational meteorological now-casting system and how PPP results compare to the network solution. According to the author, passing weather fronts can be analyzed much better by considering the information provided by GNSS derived tropospheric wet delays because this data is directly influenced by changes in humidity. It was discovered that the accuracy of the PPP ZWD estimates is worse due to several effects (satellite clock errors, biases, no ambiguity resolution), but independence from the reference station data will significantly shorten the latency of the results (few min), and provide the regional/national weather service to enhance the prognosis in the numerical forecast model.

Guo *et al.* (2016) investigated the benefits of multi-GNSS for PPP. Firstly, orbit and clock consistency tests (between different providers) were performed for GPS, GLONASS, Galileo and BeiDou. In general, the differences of GPS are, respectively, 1.0–1.5cm for orbit and 0.1ns for clock. It was discovered that consistency of GLONASS is worse than GPS by a factor of 2–3, i.e. 2–4cm for orbit and 0.2ns for clock. However, the corresponding differences of Galileo and BeiDou are significantly larger than those of GPS and GLONASS, particularly for the BeiDou GEO satellites. Galileo as well as BeiDou IGSO/MEO products have a consistency of 0.1–0.2m for orbit, and 0.2–0.3ns for clock. As to BeiDou GEO satellites, the difference of their orbits reaches 3–4m in along-track, 0.5–0.6m in cross-track, and 0.2–0.3 m in the radial directions, together with an average RMS of 0.6ns for clock. Finally, kinematic PPP tests were conducted to investigate the contribution of multi-GNSS and higher rate clock corrections. As expected, the positioning accuracy as well as convergence speed benefit from the fusion of multi-GNSS and higher rate of precise clock corrections. The multi-GNSS PPP improves the positioning accuracy by 10–20%, 40–60%, and 60–80% relative to the GPS-, GLONASS-, and BeiDou-only PPP. The usage of 30s interval clock products decreases interpolation errors, and the positioning accuracy is improved by an average of 30–50% for the all the cases except for the BeiDou-only PPP.

Seepersad and Bisnath (2014) investigated the performance of the standard-PPP technique in static and kinematic mode using one week datasets collected from 300 IGS stations from 1-7 July2012. Dual frequency ionosphere-free combination of GPS measurements was used together with the IGS 5-minute final orbits and 30-second clock correction information. The tropospheric delay was estimated as part of the adjustment process and no integer PPP ambiguity resolution was attempted in their investigation. The authors concluded that PPP in static mode could provide positional accuracy of 7 and 13 mm in the horizontal and vertical components, respectively. Using such a 24-hour dataset. In kinematic mode, the conservative accuracy of the horizontal positioning component was 46 mm, and 72 mm in the vertical component.

In view of the above literatures, this study attempts the comparative evaluation of Baselines solution, Network solutions and Precise Point Positioning post-processing strategies in view of determining their level of performance in terms of accuracy and identifying pitfalls that can limit the accuracy of any of the approach solution in point positioning

CHAPTER THREE
MATERIALS AND METHODS

3.1 Data collection

To achieve the objective of the study, Table 3.1 presents the various dataset, the format in which were obtained, sources and their respective purpose in the study.

Table 0.1: Dataset and Sources utilized in the study

S/N	Data Type	Format	Date	Source	Purpose
1	Nigeria primary trigonometric station data	DBF file	-	Office of the Surveyor General of the Federation (OSGOF)	Determination of the geometrical status of the existing Nigeria primary triangulation network.
2	GNSS Campaign data Observation data	Trimble Receiver proprietary format	October 2010- April 2011	Office of the Surveyor General of the Federation (OSGOF)	Develop first-order GNSS solutions.
3	First-Order Station Initial Coordinates Solution in WGS84 reference to ITRF2008	Excel format (.xls)	-	Office of the Surveyor General of the Federation (OSGOF)	Served as reference stations
3	Precise Ephemeris	SP3 files	October 2010- April	IGS (International GNSS Service) http://igscb.jpl.nasa.gov/	For post-processing of GNSS campaign observations

2011	products/

3.2 Data Processing Software

1. Trimble Total Control: The GNSS observation was processed using Trimble Total Control software 2.73 developed by Trimble Navigation Ltd that came into existence in 1978 with hundreds of patents. The final version 2.73 was released in 2003 available in many different languages. Trimble Total Control software provides exceptional advanced geodetic control capability plus powerful processing and tools to enable large data sets to be processed extremely quickly, with extensive analysis and reporting ideally suited to handling large GPS networks and long baselines providing an intuitive and fast solution for survey and geodetic data analysis (Trimble, 2015).

2. GNSS-LabTool (gLAB): An advanced educational and professional package used for this study to provide the precise point solution developed under an European Space Agency (ESA) Contract by the research group of Astronomy and Geomatics (gAGE) from the UniversitatPolitecnica de Catalunya (Sanz*, et al;* 2012).

3. The RTKGET automatic FTP script feature of the RTKLIB version 2.4.2 was used to stream the IGS precise GNSS products; satellite orbit, clock information data, differential Code Biases (DCBs) data for Satellites and Receivers.

3.3 Data Preparation and Processing

Within the scope of this project, the GNSS data processing included the review and cataloguing of collected data files, processing phase measurements to determine baseline vectors and unknown positions, performing adjustments and transformations of the processed vectors and positions. Figure 3.1 shows the processing stages adopted for the study.

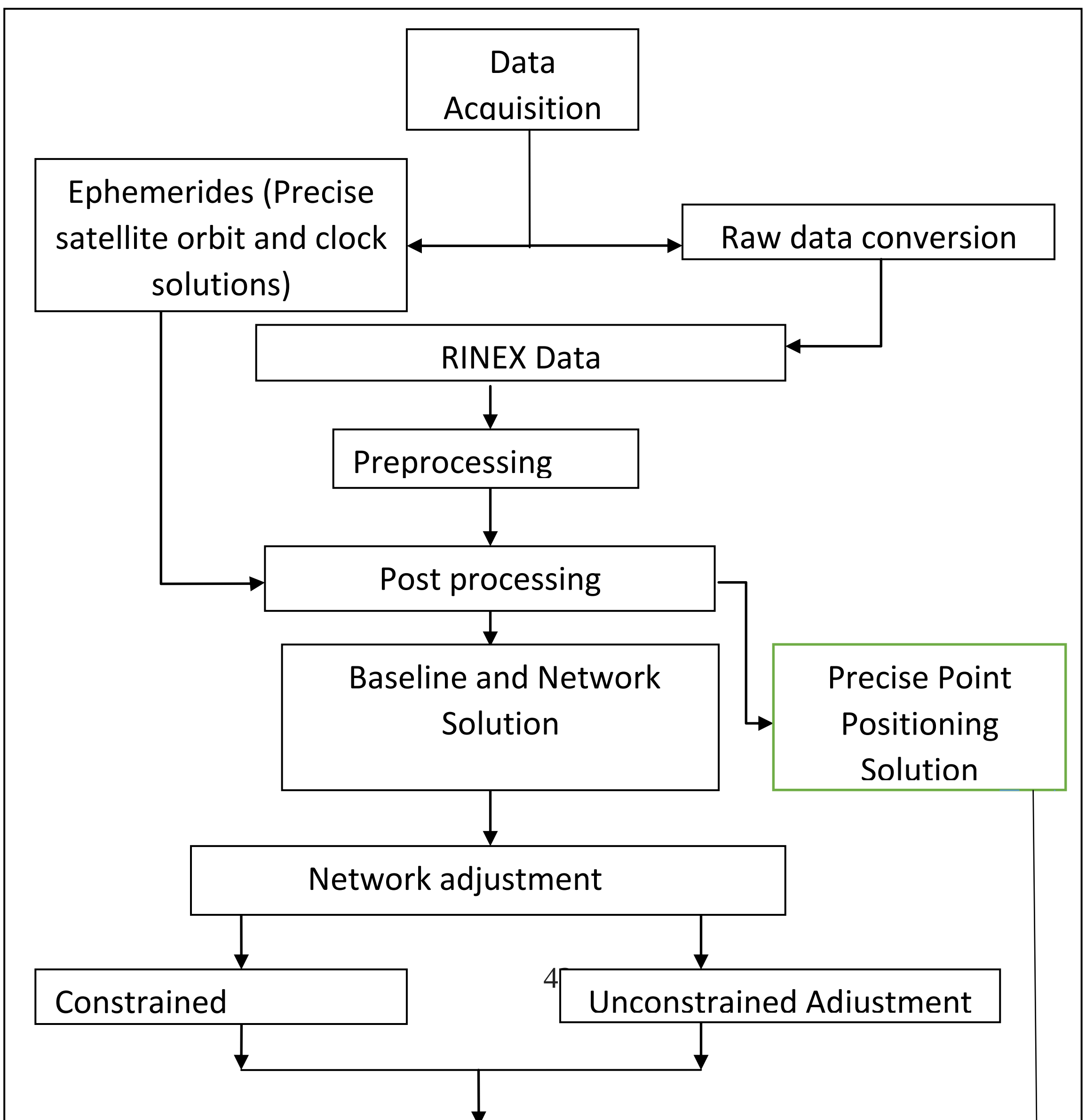


Figure 0.1: Flow chart of the processing stage

The following were applied for the processing of the GNSS observation data:

3.3.1 RINEX data conversion

The geodetic GPS software converted the raw observed GNSS data to Receiver Independent Exchange format (RINEX) format for processing. Thus, a GPS campaign was carried out from October 2010 to April 2011. A total of 60 stations were observed for a period of 48 hours GPS observations were carried out on some existing Nigerian Primary Triangulation stations, while some stations were re-established.

The campaign GNSS raw data were grouped into their respective year of observation and further grouped into sessions according to days of observation as shown in the Table 3.2.

Table 0.2: Summary of GNSS observation data used in the study

Year of Observation	Month	Days of Observation Sessions
	October	14, 17, 20, 23, 26 and 30
2010	November	1, 4, 7, 10, 14, 17, 20, 21, 24, and 28
	December	1, 4, 7, 10, 13, 16 and 19
	January	22, 25, 26, 28, 29, and 31
2011	February	4, 7, 10, 13, 16, 20, 23 and 27
	March	2, 5, 8, 10, 13, 16, 20, 23, and 27
	April	2 and 5

In Table 3.2, there are a total of 47 sub-sessions spanning for a period between October, 2010 and April, 2011.

3.3.2 Processing parameters

Geodetic parameters are estimated at one station relative to another. This mode of positioning uses network approach to define position of the unknown. High accuracy differential positioning requires one reference GPS receiver to be located at a base station whose coordinates are known, while the second user GPS receiver simultaneously tracks the same satellite signals. When the carrier phase data from the two receivers is combined and processed, the user receiver's coordinates are determined relative to the reference receiver.

The following steps were adopted to obtain the baseline solution, the a priori coordinates, ephemeris file used in processing, data file names, antenna height and possible offsets, etc., were prepared. The use of dual frequency observations (both L1 and L2 frequencies) eliminates the major part of ionospheric effect on the signal, thus improving the accuracy of positioning.

Table 3.3 summarized the processing parameters used in the post processing of the GNSS observational raw data using Trimble Total Control software.

Table 0.3: Processing Parameters used with Trimble Total Control software.

PARAMETERS	
Ephemeris	IGS Final Orbits
Observational Time	30seconds Observational Sampling rate Interval
Processing Mode	Static Mode
Solution Mode	Iono-Fixed Type
GNSS Observable	Double Differencing Carrier Phase
Elevation Cut-off Angle	10^o
Solution Type	Iono-Free Fixed
Frequencies	L1 and L2

Reference Frame	IITRF2008

Since Precise Point Positioning (PPP) is a zero-differencing technique that uses GNSS data collected from single receiver with precise satellite ephemeris and clock data, takes into account corrections like carrier phase wind-up, satellite antenna phase offset and tidal effects (Isioye *et al.*, 2018). These rigorous error corrections are required to achieve ultimate accuracy. Table 3.4 summarized the IGS products and error models employed using gLAB software for PPP Processing.

Table 0.4: PPP Processing Parameters using gLAB Software.

IGS PRODUCTS	**USED**
IGS Orbit	IGS Final
IGS Satellite Clocks	IGS Final
ERROR MODEL USED	
Receiver Antenna Phase Center Correction	YES(ANTEX)
Receiver Antenna Phase Center Correction	Yes
Receiver Antenna Reference Point Variation	Yes(RINEX)
Satellite Antenna Phase Center Offset Correction	Yes
Satellite Antenna Phase Center Offset Variation	Yes
Relativistic Clock Correction(Orbit	Yes

Eccentricity)	
Tropospheric Correction	Neil Mapping
Ionospheric Correction	Yes (IONEX)
Solid Tides Correction	Yes
Phase Wind-Up Correction	Yes
P1-P2 Correction	Yes

3.5 Network Design

Network design, included the determination of the number and location of control stations for network constraints, the selection of new project station locations, and the relative dispersion of network observations as shown in Figure 3.2 and Figure 3.3. The networks contained only closed loops. Each station in the network was connected with at least two different independent baselines. 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. All of the control stations to which the network will be constrained to must have its positions accurately known before it can serve as a constraint else it will likely add unacceptable levels of uncertainty to the network.

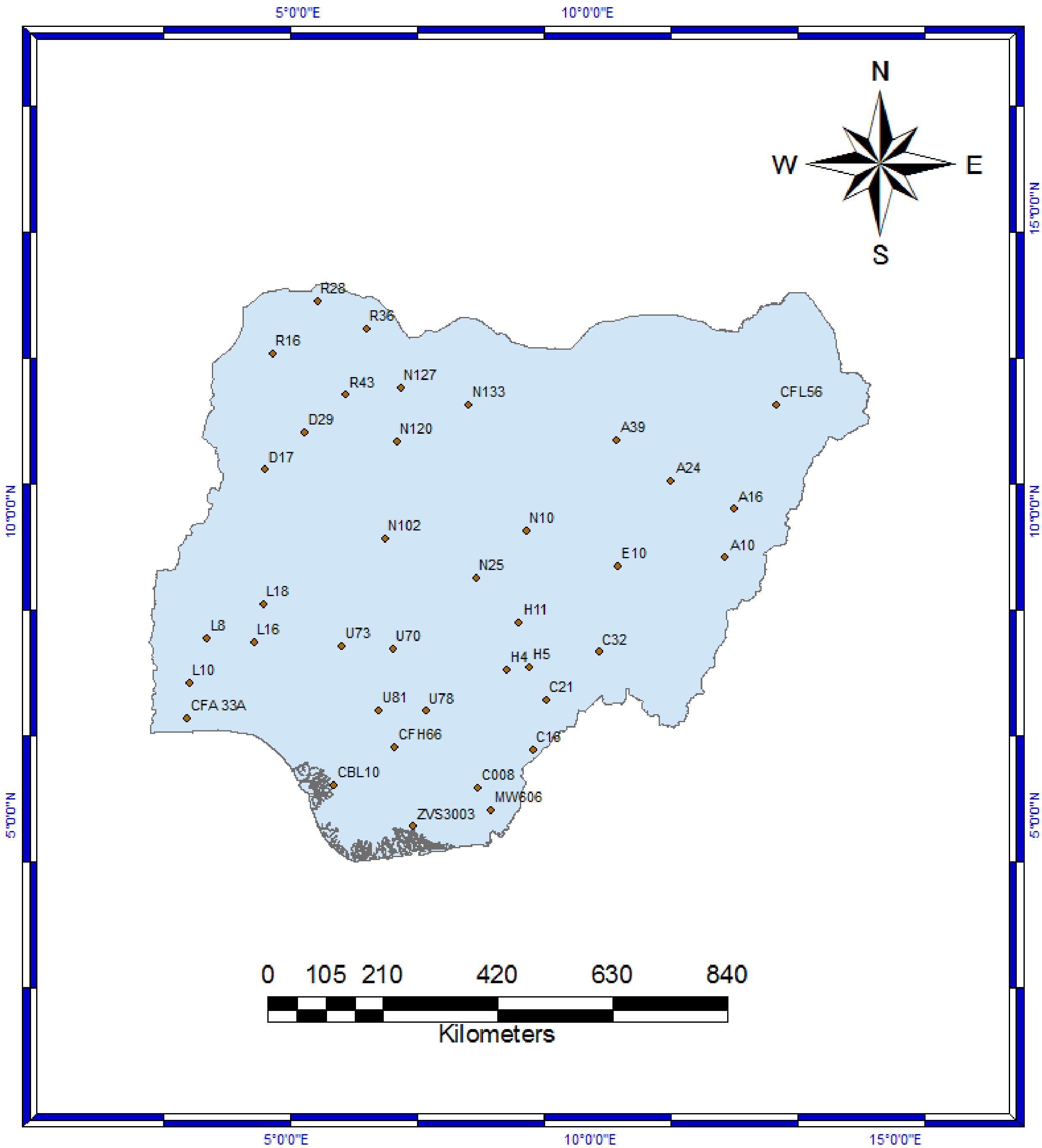


Figure 0.2: First Order Trigonometry Stations of Nigeria

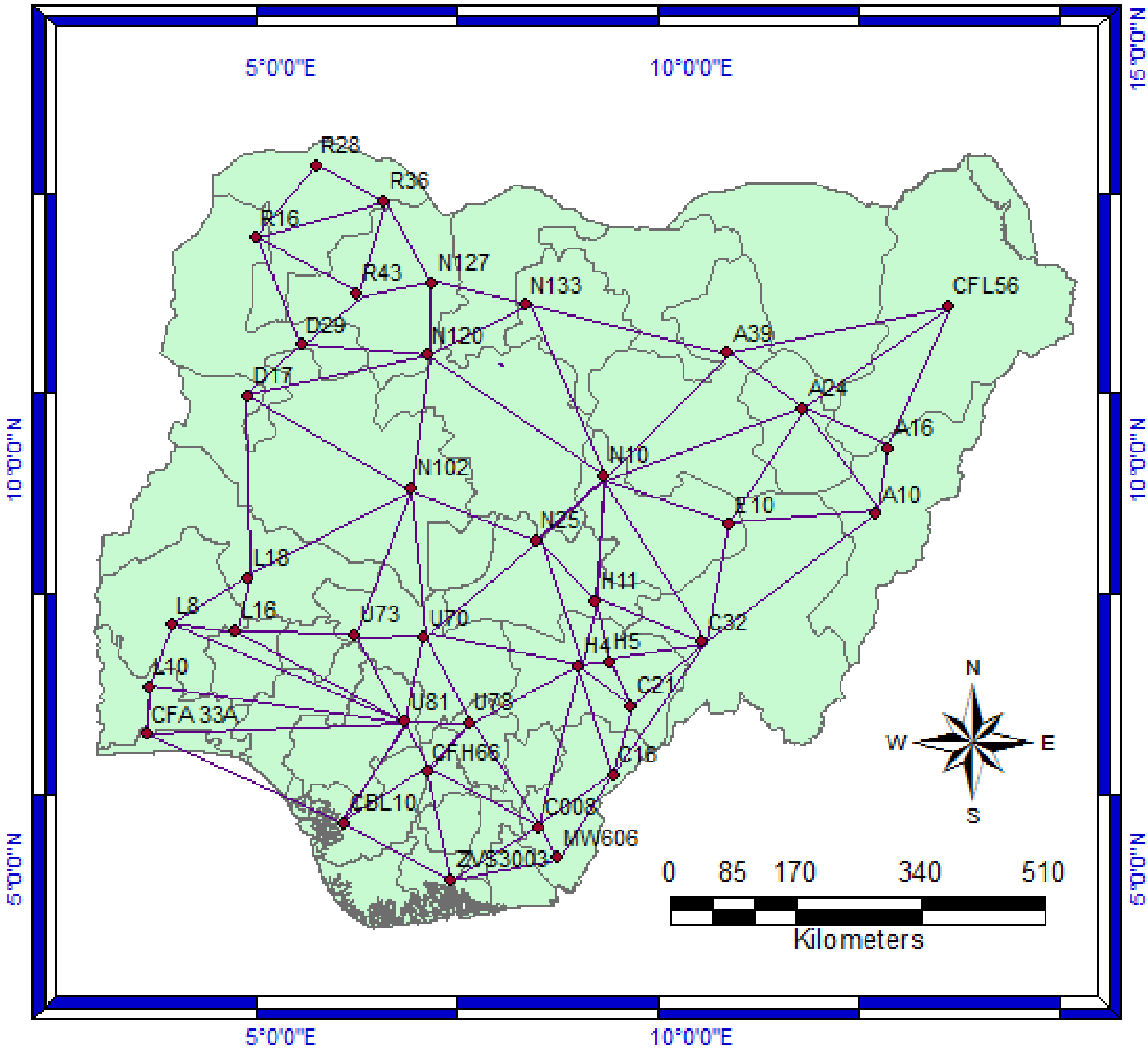


Figure 0.3: Some First-Order Network in Nigeria

3.5.1 Well-conditioned triangles in the old primary network

The accuracy of a triangulation system is greatly affected by the arrangement of triangles in the network layout and the magnitude of the angles in individual triangles. In any triangle of a triangulation system, the length of one side is generally obtained from computation of the adjacent triangle. The error in the other two sides if any, will affect the sides of the triangles whose computation is based upon their values. Due to accumulated errors, entire triangulation system is thus affected thereafter.

3.6 Network Adjustment

Two types of network adjustment were carried out after quality check of the network. They include:

1. Unconstrained network adjustment
2. Constrained adjustment

3.6.1 Unconstrained adjustment

3-D unconstrained adjustment was carried out to evaluate the internal observations of the GNSS data. During this stage, minimum number of stations obtained from OSGoF were held in other to develop values that are close to the final values. The network turned out to be a closed system, thus easing checks on the geometric closures and the consistency of the observations by the software. After the adjustment was completed, the software generated report of the statistical results of the adjustment such as the standard deviation, error ellipse, and observational residual. The statistical result was used to check the quality of the network. However, even though a minimally constrained adjustment checks the data for internal consistency, it may also fail to identify the presence of systematic errors and mistakes. Hence the need to perform constrained adjustment among other reasons.

3.6.2 Constrained adjustment

Constrained adjustment was the final step of the least squares adjustment. For the constrained adjustment, controls of coordinate values that are current and meets the reference network accuracy standards obtained from OSGoF of the most recent epoch were held fixed. These controls were of the same order of accuracy with the intended study referenced to WGS84, adjusted to ITRF2008 reference ellipsoid. The controls were evenly spaced throughout the survey project in the software in a manner that no points of interest to be determined is outside the area encompassed by the exterior reference stations

With the coordinates of these known control points held fixed, the adjustment was made to the observations. Reports by the software was also generated and the same statistical measures as that of the unconstrained adjustment.

3.7 Developing points solution using different approaches

Three positioning approaches were considered in providing points solution for GNSS positioning. The solutions characterized with long data sets removes the effect of the GNSS constellation geometry on the results, thus making them fully comparable with each other as found in (Wells *et al.*, 1999; Biagi and Sans`o, 2009; Caldera, 2010; Petovello., 2011; Chen, 2015; Ovstedal, *et al.*, 2016; Liang, *et al.*, 2018):

i. Reducing the observation campaign to simultaneous data observed from pairs of stations (baselines), and then combining these baselines into a network.
ii. Take the GPS observations observed simultaneously at all stations directly into a network adjustment where all the coordinates of the network are present as unknowns.
iii. Precise Point Positioning (PPP) approach.

Adopting the above mentioned approaches, the GNSS observations were grouped accordingly whilst keeping all processing parameters because in principle, this two approaches are based on collecting simultaneous measurements at both the reference station and remote receivers for a certain period of time, which, after processing, yield the coordinates of the unknown point. The same stations data for 24hr period was processed again using the GPS PPP approach. Twenty-four hour (24hr) of all stations was used because the quality of the position estimates is very dependent on the observation session length. As a general rule, a minimum of one hour is required for the horizontal solution from a standard-PPP static processing to converge to 5cm (Seepersad and Bisnath 2014). However, unlike the first two approaches common mode errors do not cancel in PPP thus error models as presented in Table 3.4 were included in the PPP solution during the processing to achieve high accuracy.

The resultant solutions of all the approaches are to be compared with published coordinate values to assess and analyze the performance of all of the above mentioned approaches.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Geometric Analysis of the Existing Network

The Nigeria geodetic controls are presented in geodetic latitude and longitudes of station points with reference to the Minna Datum (SURCON, 2003). The network as shown in Figure 3.3 consists of inter-connected stations distributed across the country obtained from Office of the Surveyor General of the Federation as provisional coordinates forming a triangulation system.

For a network to fulfil its basic goal as a strong and reliable reference framework, the individual figures should be well shaped with stations evenly spaced out as much as possible and all adjacent pairs of stations in the network should preferably be connected by direct stage.

Fifty-six (56) stations made up the network. The triangles generated are Eighty-two (82) in number; numbered 1-82 as presented in Table 4.1, represented in geographical coordinates. For complete result of Table 4.1 see APPENDIX 1.The stations have a unique identifier made by OSGoF in order not to have confusing nor conflicting numbers. **When designing Nigerian primary triangulation network, a number of geometric conditions were made which include the following:**

i. **The number of stations in the Figure**

ii. **The ratio of length of side to each other**

iii. **The number of lines in the Figure**

iv. **The number of line observed in two direction, and**

v. **The number of lines observed in one direction.**

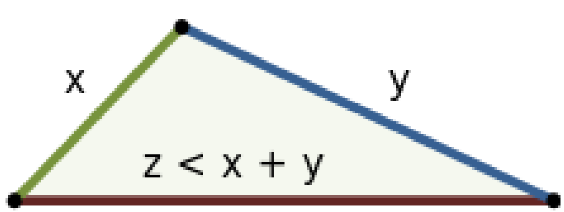
Table 0.1: Sample Details of the Old Triangulation Network

Stations	**LAT. (Deg)**	**LONG. (Deg)**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
R43	12.02125	5.894593	456.44	157.4386	79.59525	R16
R16	12.69258	4.656074	335.118	130.0843	56.71921	R28
R28	13.57862	5.396466	348.972	185.1564	43.68554	R43
R43	12.02125	5.894593	456.44	185.1564	43.80453	R28
R28	13.57862	5.396466	348.972	105.1549	102.536	R36
R36	13.13372	6.224632	412.254	131.3217	33.65945	R43
R43	12.02125	5.894593	456.44	131.3217	46.75124	R36
R36	13.13372	6.224632	412.254	129.6484	67.50974	N127
N127	12.14852	6.816151	754.644	103.5925	65.73902	R43
N127	12.14852	6.816151	754.644	129.6484	33.51273	R36
R36	13.13372	6.224632	412.254	289.9146	21.48599	K001
K001	12.0143	8.566396	482.988	195.427	125.0013	N127

N133	11.87248	7.95495	618.726	130.557	9.373597	N127
N127	12.14852	6.816151	754.644	195.427	17.77548	K001
K001	12.0143	8.566396	482.988	69.92872	152.8509	N133
N133	11.87248	7.95495	618.726	69.92872	85.51077	K001
K001	12.0143	8.566396	482.988	254.0796	15.65646	N10
N10	9.785181	8.91557	1421.24	258.237	78.83277	N133
N10	9.785181	8.91557	1421.24	254.0796	50.47233	K001
K001	12.0143	8.566396	482.988	230.865	70.59435	A042
A042	10.96978	10.3495	585.132	207.8971	58.93332	N10
A042	10.96978	10.3495	585.132	230.865	9.021411	K001
K001	12.0143	8.566396	482.988	221.7417	99.8473	A39
A39	11.28886	10.4174	474.746	36.75285	71.13129	A042

With triangle as the basic shape of the geometry, the network's geometry status is analyzed on how well the triangles are conditioned in the network given the derived angles and length of sides. A well-conditioned triangle will have it edges far enough and vertex clearly identified while ill-conditioned triangle can have ambiguity in vertex position leading to a weakness in that part of the network or wrong measurements. Another way of identifying an ill-conditioned triangle is applying the principle of triangle inequality.

The triangle inequality states that the sum of the lengths of any two sides of a triangle must



be greater than or equal to the length of the third side. That sum can equal the length of the third side only in the case of a degenerate triangle, one with collinear vertices (i.e. $Z < X + Y$ as shown in Figure 4.1). It is not possible for that sum to be less than the length of the third side, (Posamentier and Lehmann, 2012).

Figure 0.1: Triangle Inequality

To identify areas of strengths and weakness in a triangle, the network's geometry was analyzed based on the above theorem because to achieve a highest accuracy from any triangulation system, the shape of the triangle should be such that any error in the measurement of angle should have a minimum effect upon the lengths of the calculated sides. Such a triangle is called well-conditioned triangle hence, the identification of the triangles having angles less than 30^o or more than 120^o. Suggestions were made on how to strengthen those parts of the network that did not meet the theorem's requirement.

Upon subjection to analyze the triangle inequality of the old triangulation network, it was observed that 56% of the triangles in the network fulfilled the condition while 44% did not suffice; 30% of the triangles an angle less than 30^o, while 5% of the triangles had its two angles out of three less than 30^o, 3% of the triangles had angles greater than 120^o and 5% of the triangles had its two angles out of three less than 30^o or more than 120^o.

Triangles with angle less than 30^0 or greater than 120^0 should not be considered during computation in order to maintain the network within the desirable degree of precision but can however be used if it’s not opposite of the side whose length is required to be computed for carrying forward the triangulation series.

Obviously, additional observations strategically positioned with respect to other adjacent points, will enhance those part of the network that did not satisfy the condition. Of course, poor location of control and line obstructions that occur due to terrain, vegetation, buildings or geography especially in the southern and eastern parts of the country where the topography was not suitable for the establishment of triangulation points due to non-availability of hills and high grounds which was one of the major limitations faced during triangulation network design preventing observations from being obtained but can however be solved using other survey technique like GNSS technique that does not require station intervisibility for connection to form a triangulation system.

Similarly, the network efficiency could be improved if a connection could be made to south-west of the survey and observations between those areas that were termed “Bad” can also improve dramatically, the overall strength in the network.

4.2 Result of the Free and Constrained Adjustment of the GNSS Network

The adjustment provided a single set of coordinates based on all the measurements acquired, as well as providing a mechanism by which baselines that have not been resolved to sufficient accuracy can be detected. The GNSS observations were processed and all the baselines of the network computed. The results of the 3D unconstrained and constrained adjustment are shown in the Figure 4.2 and 4.3 respectively.

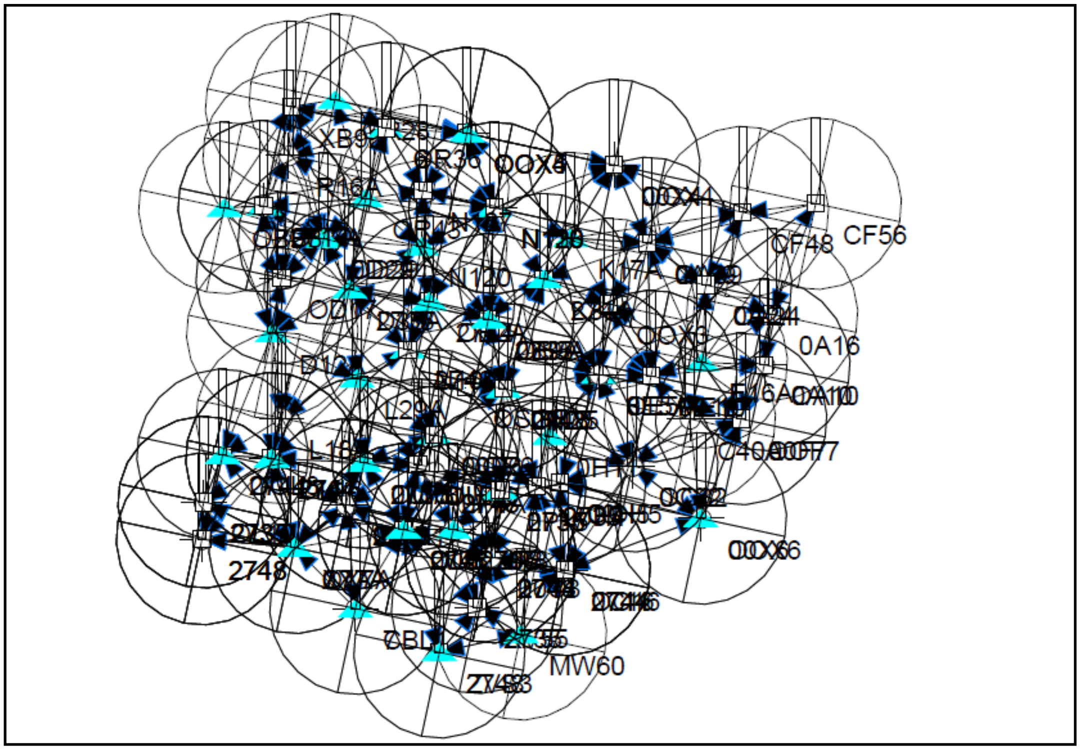


Figure 0.2: Plot of the Unconstrained First-Order Adjusted Network of Nigeria

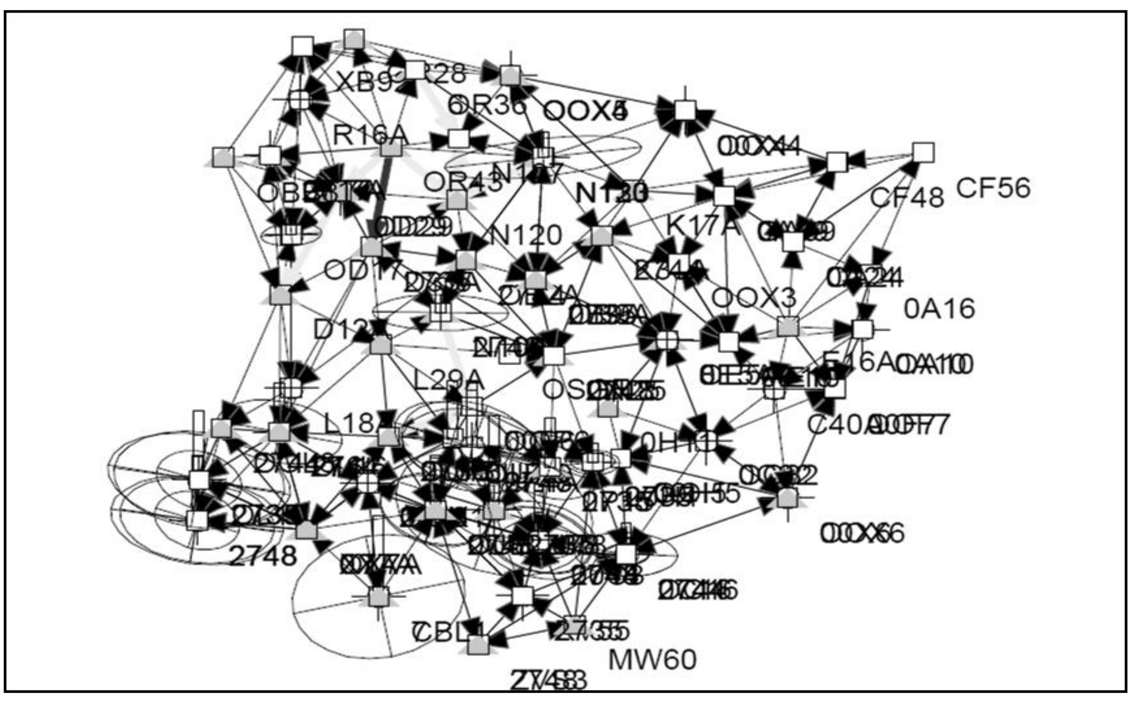


Figure 0.3: Plot of the Constrained First-Order Adjusted Network of Nigeria

During the adjustment, the following baselines 00H11-00H5, 0C32A-0E5A, 00X3-0E16A, and XB92-0R43 did not pass the quality check due to antenna height error.

4.3 Differences in Solutions Realized by the Approaches

In order to numerically compare the results provided by the three approaches as explained in chapter three, network of 52 (fifty-two) stations was used. For the sake of clarity and easy referencing, the following naming convention was adopted to represent the different solutions realized by the different approaches namely:

Approach 1: to represent the reduced observation campaign observed from pairs of stations combined to form a network

Approach 2: to represent the network obtained when the GPS observations were observed simultaneously at all stations.

Approach 3: to represent Precise Point Positioning (PPP) solutions

4.3.1 Final coordinate solution of the approaches

Table 4.2 presents the final stations coordinate solution based on the approach of reduced observation campaign observed from pairs of stations combined to form a network in the East, North and Height component.

Table 0.2: Final Station Coordinate Solution of Approach 1

S/N	Stations	EAST (m)	NORTH (m)	HEIGHT (m)
1	00F7	813049.4966	936901.4917	1565.57487

2	00H4	456217.7476	824921.8684	362.6327374
3	00H5	496880.1497	830363.3931	300.8614709
4	00L8	569062.0805	879503.4328	465.0494665
5	00P6	277358.8568	915055.9303	393.9391745
6	00X1	786016.5204	794757.8468	370.6914507
7	00X3	586754.8238	1141720.746	675.2890242
8	00X4	597235.2627	1385129.306	386.6136874
9	00X6	741323.0748	765139.4343	1793.494655
10	0A10	195690.1231	1031363.938	286.9794435
11	0A16	212422.92	1120382.204	786.5242444
12	0A24	755997.1735	1173133.68	624.3709539
13	0A39	654702.7315	1248293.911	495.4689678
14	0B2A	269920.5432	1150677.077	713.1313394
15	0C16	502948.0494	678356.1122	628.4396895
16	0C32	623559.0122	857731.908	391.5132139
17	0D29	741959.4434	1259801.567	527.2741248
18	0E5A	567053.9389	1020181.071	607.6613707
19	0H11	478322.9039	911692.2477	259.3779347
20	0L10	538073.7074	796359.2538	198.7350884
21	0L16	654752.4036	873958.3134	524.7273116
22	0N25	399684.7833	994841.6365	592.1008578
23	0P15	390756.6741	818069.3609	293.7562102
24	0P4A	277386.1327	850693.8294	193.377417

S/N	Stations	EAST (m)	NORTH (m)	HEIGHT (m)
25	0R28	759326.3715	1502398.75	370.0408672
26	0R36	199085.7425	1453580.327	431.8409703
27	0R43	815197.6665	1330562.461	477.763112
28	0R7A	637780.6015	1315021.097	291.1892883
29	0U70	247793.6036	863715.3102	437.4351222
30	0U73	816264.9506	868103.2718	677.0547352
31	0U78	308068.1141	749375.7219	424.1459919
32	0U81	222078.3562	750924.7164	215.8662665
33	0X7A	696301.2146	718902.4663	59.44336043
34	C40A	723796.8735	938499.6929	257.9686061
35	CBL1	802743.1081	612827.9838	23.05505964
36	CF48	821310.2188	1298922.282	393.9110781
37	CFA3	535711.277	732516.5615	71.79768749
38	D12A	653662.786	1094605.751	279.3344896
39	D35A	788075.1181	1171148.881	375.9124086
40	K17A	519662.26	1257505.172	484.8959032
41	K38A	472500.3912	1186886.16	905.9235433
42	L18A	671479.1387	943998.9668	430.4561809
43	L29A	803814.5502	1014428.751	151.34742
44	MW60	426718.1041	566191.3097	105.3085138
45	N102	232076.165	1066461.366	471.866097
46	N120	258806.2324	1246222.028	594.7141506

S/N	Stations	EAST (m)	NORTH (m)	HEIGHT (m)
47	N127	262342.5405	1343929.896	776.3388601
48	N133	386174.7428	1312666.161	639.4836836
49	0R16	679826.0834	1403710.223	355.4400468
50	XB92	682602.7343	1489850.44	257.4546748
51	XV55	349186.0352	614304.0091	155.1420534
52	ZVS3	283500.2579	536171.133	35.33090564

Table 4.3 presents the final stations coordinates based on the network obtained when the GPS observations were observed simultaneously at all stations in the East, North and Height components.

Table 0.3: Final Coordinates Solution based on Approach 2.

S/N	Station	EAST (m)	NORTH(m)	HEIGHT (m)
1	00F7	813049.3003	936901.2918	1565.067065
2	00H4	456218.0376	824921.9552	362.151854
3	00H5	496880.0034	830364.3135	300.9254701
4	00L8	569061.7845	879503.2145	464.6891707
5	00P6	277357.9833	915055.3794	394.4726742
6	00X1	786017.4582	794757.1426	370.1459436
7	00X3	586754.7997	1141720.246	674.7722459
8	00X4	597235.0484	1385128.72	386.8196965
9	00X6	741322.7845	765138.9837	1793.503113

S/N	Stations	EAST (m)	NORTH (m)	HEIGHT (m)
10	0A10	195689.3096	1031363.09	287.0636174
11	0A16	212422.3233	1120381.897	786.848526
12	0A24	755996.4046	1173132.783	623.6379539
13	0A39	654702.5251	1248293.619	495.1055702
14	0B2A	269919.7971	1150676.949	713.5283376
15	0C16	502947.9012	678355.4372	628.4803056
16	0C32	623557.9634	857731.0795	391.8929435
17	0D29	741959.1098	1259801.138	526.4162658
18	0E5A	567054.237	1020181.345	608.4359
19	0H11	478323.6622	911693.1523	259.3762295
20	0L10	538073.4801	796358.5164	198.9321831
21	0L16	654751.2976	873957.6196	524.6749096
22	0N25	399685.3988	994841.9701	591.1689859
23	0P15	390756.0703	818068.8122	293.5250838
24	0P4A	277385.3317	850693.4846	194.6523028
25	0R28	759326.1413	1502398.436	370.4856028
26	0R36	199085.1905	1453579.991	431.1475315
27	0R43	815197.0034	1330562.004	476.9143973
28	0R7A	637781.3194	1315021.313	291.5279884
29	0U70	247793.6442	863714.627	438.0700842
30	0U73	816264.3435	868103.0459	677.0047351
31	0U78	308068.2267	749375.1364	423.6332514

S/N	Stations	EAST (m)	NORTH (m)	HEIGHT (m)
32	0U81	222077.6653	750924.1943	215.7115237
33	0X7A	696300.6758	718901.7174	59.05244859
34	C40A	723796.444	938498.7802	257.3686275
35	CBL1	802742.5337	612827.7434	22.76116193
36	CF48	821311.1693	1298922.513	393.4712477
37	CFA3	535710.8604	732516.983	70.89698725
38	D12A	653662.14	1094605.236	278.9036312
39	D35A	788075.2836	1171149.289	375.8614613
40	K17A	519662.8402	1257505.985	484.8154979
41	K38A	472500.6592	1186886.415	906.5545464
42	L18A	671479.9417	943999.6262	429.5611222
43	L29A	803814.7609	1014429.042	151.3779814
44	MW60	426718.2791	566190.4989	104.6413436
45	N102	232075.6147	1066460.989	470.9637067
46	N120	258805.6276	1246221.495	594.162251
47	N127	262343.5861	1343930.882	775.93475
48	N133	386174.4845	1312665.766	639.5950807
49	0R16	679825.4398	1403709.624	354.5462858
50	XB92	682603.1223	1489850.724	257.5105588
51	XV55	349186.1946	614304.8712	155.0345083
52	ZVS3	283500.8378	536171.6044	34.60914039

Table 4.4 presents the final stations coordinates solution based on Precise Point Positioning (PPP) approach in the East, North and Height components.

Table 0.4: Final Stations Coordinate solution based on Approach 3

S/N	Stations	EAST (m)	NORTH (m)	HEIGHT (m)
1	00F7	813049.5	936901.7	1565.646
2	00H4	456217.6	824921.9	362.6899
3	00H5	496880.1	830363.4	300.3621
4	00L8	569062.2	879503.4	465.1636
5	00P6	277358.7	915056.2	394.0427
6	00X1	786016.5	794757.6	370.9004
7	00X3	586754.7	1141721	675.4353
8	00X4	597235.3	1385129	386.3966
9	00X6	741322.8	765139.3	1793.584
10	0A10	195690.1	1031364	286.954
11	0A16	212423.3	1120382	786.8597
12	0A24	755997.2	1173134	624.4373
13	0A39	654703	1248294	495.4566
14	0B2A	269920.6	1150677	713.4488
15	0C16	502948.1	678355.9	628.6334
16	0C32	623558.7	857731.9	391.7728
17	0D29	741959.4	1259802	527.1663
18	0E5A	567054	1020181	607.6343

S/N	Station	EAST (m)	NORTH(m)	HEIGHT (m)
19	0H11	478322.7	911692.3	259.4748
20	0L10	538073.7	796359.2	199.1412
21	0L16	654752.3	873958	524.7143
22	0N25	399685.1	994841.7	591.8032
23	0P15	390756.7	818069.5	293.9882
24	0P4A	277386.1	850693.9	193.5313
25	0R28	759326.1	1502398	370.1895
26	0R36	199085.5	1453580	431.8218
27	0R43	815197.7	1330562	477.9508
28	0R7A	637780.7	1315021	291.3112
29	0U70	247793.6	863715.3	437.4049
30	0U73	816265	868103.3	677.1308
31	0U78	308068.1	749375.7	424.2841
32	0U81	222078.3	750924.8	215.7293
33	0X7A	696301.3	718902.4	59.41775
34	C40A	723796.9	938499.5	257.9576
35	CBL1	802743.1	612827.8	23.08445
36	CF48	821310.3	1298922	393.9872
37	CFA3	535711.4	732516.7	71.75123
38	D12A	653662.8	1094606	279.4086
39	D35A	788075.1	1171149	375.8191
40	K17A	519662.4	1257505	484.9193

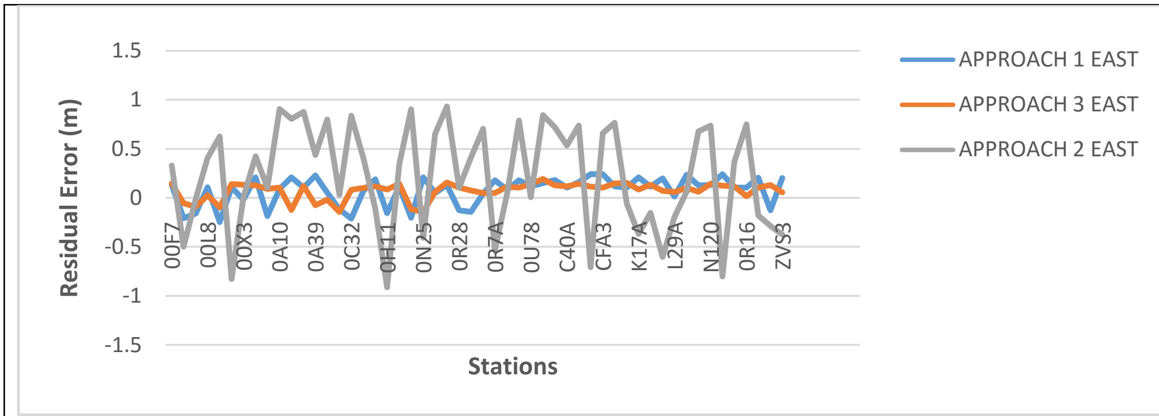
S/N	Station	EAST (m)	NORTH(m)	HEIGHT (m)
41	K38A	472500.4	1186886	905.8248
42	L18A	671479.3	943999	430.465
43	L29A	803814.5	1014429	151.3035
44	MW60	426718.2	566191.2	105.3681
45	N102	232076.2	1066461	471.58
46	N120	258806.2	1246222	594.8078
47	N127	262342.7	1343930	776.6396
48	N133	386174.7	1312666	639.7022
49	0R16	679826.2	1403710	355.2993
50	XB92	682602.8	1489850	257.445
51	XV55	349185.8	614303.9	154.9103
52	ZVS3	283500.4	536171	35.36931

4.3.2 Performance analysis of the approaches

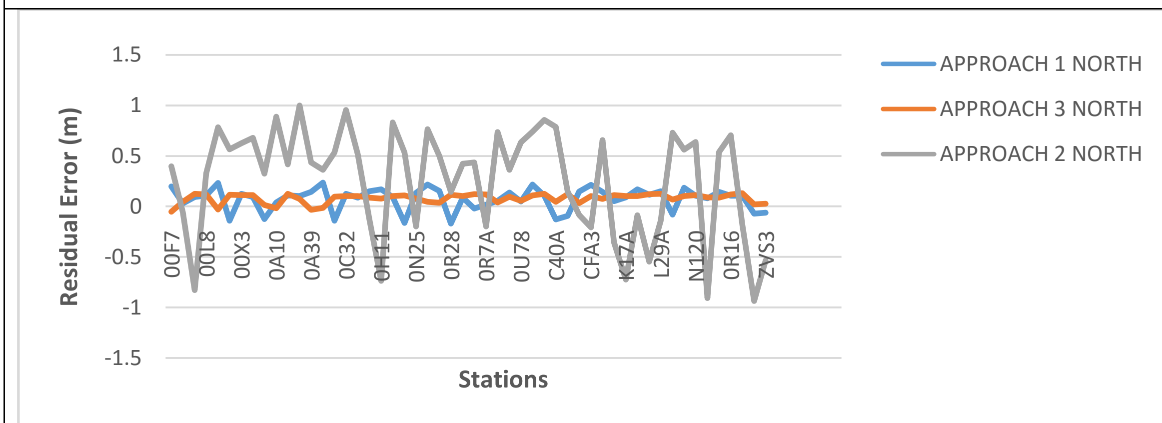
To determine the performance level of the approaches, comparison of each approach solutions was made with the ‘standard’ coordinates of same stations. These ‘standard’ coordinates of the stations obtained from OSGoF were used as points of reference for the comparison. Fig 4.1 (a), (b) and (c) shows the difference between the approaches and the standard coordinates in the horizontal and vertical components respectively.

It is evident from Figure 4.4 (a), (b), and (c) that Approach 2 had the greatest residual errors in the East, North and Height component with respect to the standard station coordinate solutions. Minimum residual error of -0.15m was seen at station k38A and maximum residual error of 0.93m was seen at station 0P4A in the East component while in the North component of Approach 2 minimum residual error of -0.05m at station K38A and maximum value of 0.98m at station 0A24.However, in the height component the minimum residual error of -0.04m was encountered at station 0C32 while the maximum value of 0.99m was seen at station 0C32.

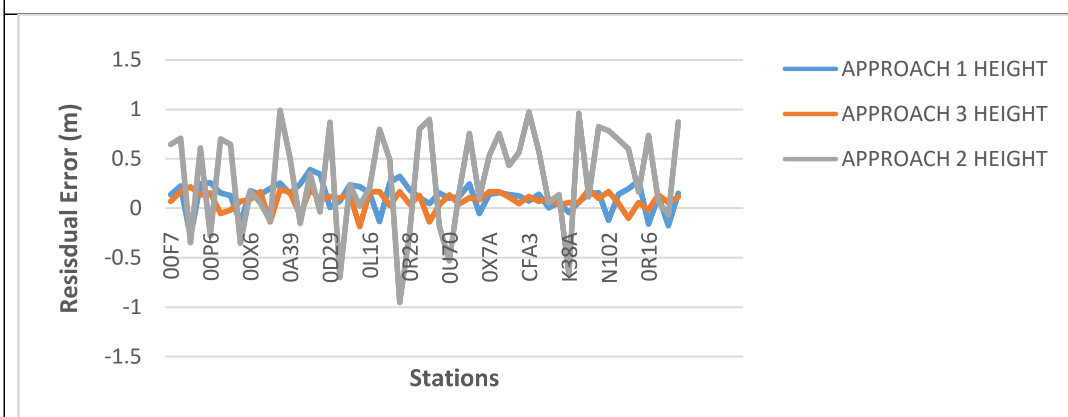
Approach 2 gave the least accurate result compared to the other approaches due to many bias factors introduced as a result of the network obtained. Approach 2 simultaneous observation combination of all stations gave rise to increase in size of network and large distance between reference station and points to be determined within the network.



(a) East Component



(b) North Component



(c) Height Component.

Figure 0.4: Discrepancies between Coordinate Solutions of the Approaches with respect to standard Solutions, (a) East Component (b) North Component (C) Height Component

As a requisite, adequate number of existing control point connections are often required in the specifications. This is to ensure strong network geometry for other points of unknown but as the inter-station distance increases, the problems of accounting for distance-dependent biases grows and, as a consequence, reliable ambiguity resolution becomes difficult to fix. The highest accuracy positions are obtained only after the estimation of the carrier phase ambiguities but the further away the rover is from the reference station, the more these errors are de-correlated, and hence not cancelled out in the differences thus degrades positioning accuracy (Tajul *et al.*, 2006).

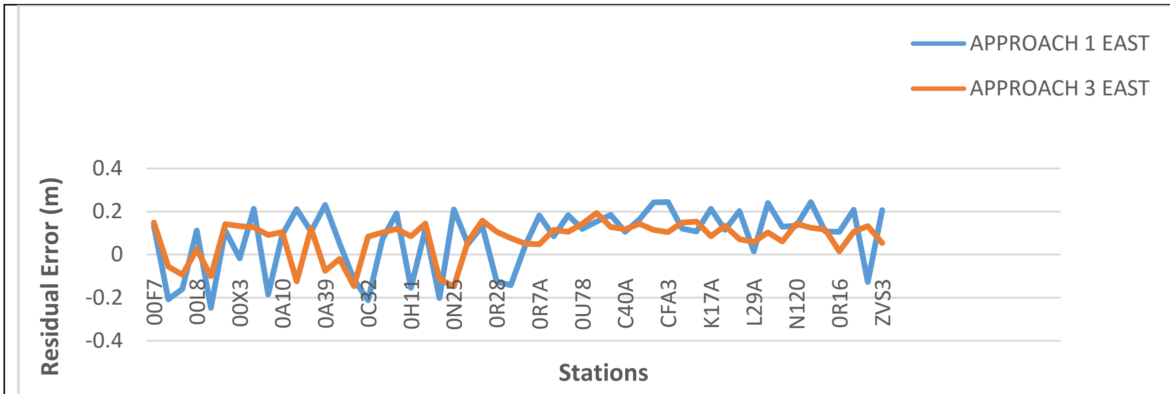
Also, due to large inter-station distances between the reference stations and the 'user' station, different metrological measures such as temperature, pressure and humidity exist at both ends. These local environmental weather condition of the GNSS receivers at various end was not taken into consideration, rather, standard atmospheric values were used during the data processing, thus, not all the effects were cancelled out during the differencing (Odiaka, 2017).The GNSS observational campaign ran into different months and season (October, 2010 – April, 2011). Although models were used to correct for atmospheric error, the models cannot adequately account for this errors since the prevailing metrological functions remained constant at the respective stations were not measured thus, the processing procedures assumes that the atmospheric conditions remained constant all through the campaign period. The model computes a constant correction at each station regardless of the season which was not the case in reality this was not the case. This considerably induced one of the biggest GNSS error; atmospheric errors in the positional values of the processed points of interest.

Furthermore, spatial configuration of a network with respect to the reference station and the user station affects accuracy of the network (Raquet*et al* 2001; Lachapelle and Alves, 2002; Alves *et al.*, 2003). Approach 2 had the configuration between the reference stations and the unknown points not well balanced with respect to each other. This technique directly adjusted all simultaneous observations where all the stations present are unknowns. It resulted to some points to be determined either at the edge or outside of the reference stations. Better performance in this regard will be seen when the points to be determined is within especially at the center of the network (Raquet*et al* 2001).

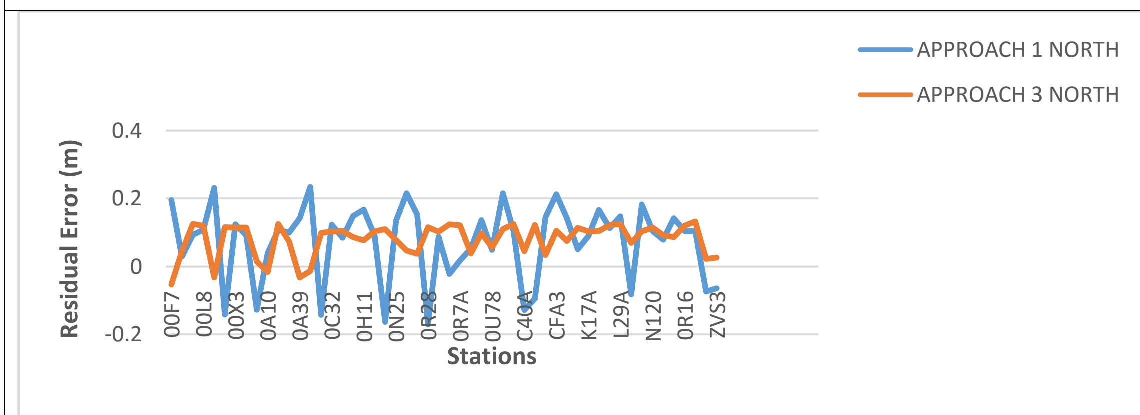
In addition, GNSS networks where many times the phase double differences are treated as observations, hypothesis of linear independence is by definition inconsistent gives rise to blunders in the solution (Caldera, 2010). In Approach 2 network solution, due to multiple observations of same stations at different days and time, there was geometric increase in error propagation among the networks during adjustment. This approach processing lead to same station having different solutions generated from different networks. In order words, same station had different position points closely clustered together by a distance that range 1m-20m apart. This introduced a lot of false redundancies that gave solution of varying magnitude for same station. This effect significantly depress the estimated covariances of station coordinates at the time of adjustment affecting positional accuracy of the point of interest.

4.3.3 Comparison analysis between approach 1 and approach 3

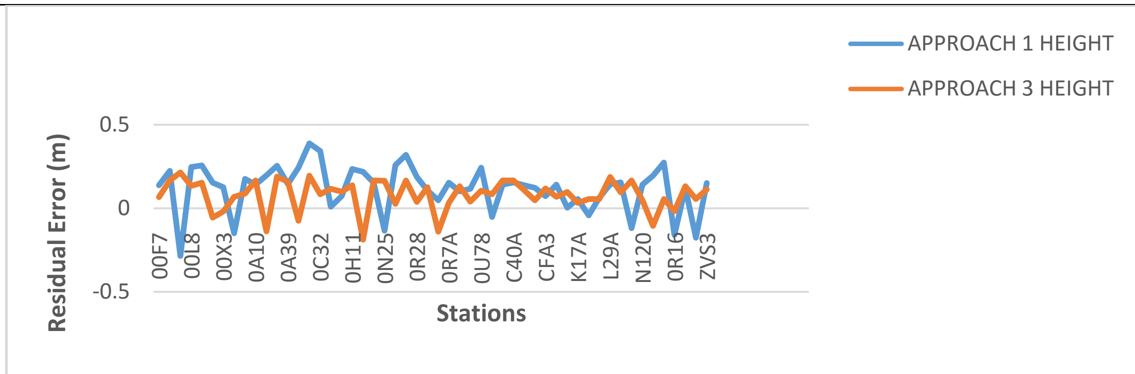
Approach 1 and Approach 3 solutions performed better with respect to standard coordinate solutions than Approach 2 as seen in Figure 4.4 (a), (b) and (c). Hence, the need to examine which of these two approaches performed better in relation to one another in terms of their respective residual errors. Figure 4.5 shows the discrepancy plot of Approach 1 and Approach 3 with respect to the standard coordinate solution.



(a) East Component



(b) North Component



(C) Height Component

Figure 0.5: Discrepancies between Approach 1 and Approach 3 Solutions with respect to standard solution, (a) East Component (b) North Component (C) Height Component

It is evident in Figure 4.5 (a), (b), and (c) that at some stations Approach 1 coordinate solutions performed better than Approach 2 solutions while in some other stations Approach 3 performed better than Approach 1 in terms of their respective residual errors. For clarity, Table 4.5, 4.6 and 4.7 presents the residual error difference between Approach 1 and 3 in the East, North and Height components of the stations.

Table 0.5: Residual Difference between Approach 1 and Approach 3 in the East Component

S/N	Stations	APPROACH 1 EAST (m)	APPROACH 3 EAST (m)	App 1-App 3 (dE)
1	00F7	0.13280071	0.148835843	-0.01603513
2	00H4	-0.207285829	-0.055705147	0.15158068
3	00H5	-0.160038513	-0.092515676	0.06752284
4	00L8	0.110501011	0.02489743	0.08560358
5	00P6	-0.247312504	-0.100164896	0.14714761
6	00X1	0.110333305	0.142033895	-0.03170059

7	00X3	-0.01772703	0.132637168	-0.11491014
8	00X4	0.211112524	0.126358624	0.0847539

S/N	**Stations**	**EAST (m)**	**NORTH (m)**	**HEIGHT (m)**
9	00X6	-0.18442949	0.090426397	0.09400309
10	0A10	0.091347349	0.104927491	-0.01358014
11	0A16	0.210012312	-0.123958273	0.08605404
12	0A24	0.107004423	0.121753927	-0.0147495
13	0A39	0.230123005	-0.074921943	0.15520106
14	0B2A	0.052222945	-0.020341974	0.03188097
15	0C16	-0.115799913	-0.145648743	-0.02984883
16	0C32	-0.211423291	0.083184719	0.12823857
17	0D29	0.071368955	0.102388765	-0.03101981
18	0E5A	0.191702304	0.11852439	0.07317791
19	0H11	-0.154359619	0.084562387	0.06979723
20	0L10	0.116430449	0.144237153	-0.0278067
21	0L16	-0.201453236	-0.113735483	0.08771775
22	0N25	0.208602552	-0.147839562	0.06076299
23	0P15	0.049003331	0.061724817	-0.01272149
24	0P4A	0.13145643	0.15834923	-0.0268928
25	0R28	-0.12649317	0.105243735	0.02124944
26	0R36	-0.14267821	0.075427253	0.06725096
27	0R43	0.04218969	0.050174356	-0.00798467
28	0R7A	0.180361168	0.048263243	0.13209793
29	0U70	0.084321804	0.114187256	-0.02986545
30	0U73	0.18221361	0.105242739	0.07697087
31	0U78	0.11786321	0.142183735	-0.02432053
32	0U81	0.15275211	0.193273683	-0.04052157
33	0X7A	0.18314532	0.128273534	0.05487179
34	C40A	0.104671261	0.118314321	-0.01364306
35	CBL1	0.162314567	0.143523147	0.01879142
36	CF48	0.241821391	0.115141363	0.12668003
37	CFA3	0.242378656	0.104893263	0.13748539
38	D12A	0.119476023	0.148326357	-0.02885033
39	D35A	0.106839261	0.152673794	-0.04583453
40	K17A	0.211527841	0.085624372	0.12590347
41	K38A	0.114326743	0.134263831	-0.01993709
42	L18A	0.201124562	0.072178691	0.12894587

43	L29A	0.015324786	0.058272415	-0.04294763
44	MW60	0.237824314	0.102387326	0.13543699

S/N	Stations	EAST (m)	NORTH (m)	HEIGHT (m)
45	N102	0.128534276	0.06134273	0.06719155
46	N120	0.132654132	0.14268392	-0.01002979
47	N127	0.242875438	0.12438274	0.1184927
48	N133	0.106543273	0.116542462	-0.00999919
49	0R16	0.105123833	0.015348932	0.0897749
50	XB92	0.207851841	0.105638713	0.10221313
51	XV55	-0.127409373	0.132064399	-0.00465503
52	ZVS3	0.205170854	0.054425379	0.15074548

Approach 1 had the minimum residual error of -0.02m at station 00X3 and a maximum residual error of 0.24m at station CFA3 in the East component while Approach 3 had a minimum residual error of -0.02m at station 0B2A and the maximum residual error of 0.15m at station D35A in the East component. Approach 1 had 42% of its stations closer to the standard while Approach 3 had 58% of its station closer to the standard values. Table 4.6 presents the residual difference between approach 1 and approach 3 in the North component

Table 0.6: Residual Difference between Approach 1 and Approach 3 in the North Component

S/N	Stations	APPROACH 1 NORTH (m)	APPROACH3 NORTH (m)	APP 1 – APP 3 (dN)
1	00F7	0.19640084	-0.052536883	0.14386396
2	00H4	0.028959657	0.04509062	-0.016131
3	00H5	0.093581782	0.125230095	-0.0316483
4	00L8	0.108829767	0.120207217	-0.0113774
5	00P6	0.231811051	-0.031557203	0.20025385
6	00X1	-0.141365823	0.115328548	0.02603728
7	00X3	0.124760555	0.114873467	0.00988709

8	00X4	0.092312424	0.114893487	-0.0225811
9	00X6	-0.126682179	0.015352423	0.11132976
10	0A10	0.040037148	-0.016253454	0.02378369
11	0A16	0.108319211	0.125345373	-0.0170262
12	0A24	0.100254632	0.073635327	0.0266193

S/N	**Stations**	**EAST (m)**	**NORTH (m)**	**HEIGHT (m)**
13	0A39	0.142568049	-0.031973232	0.11059482
14	0B2A	0.23465895	-0.013536375	0.22112258
15	0C16	-0.142311809	0.098753857	0.04355795
16	0C32	0.123887704	0.103252687	0.02063502
17	0D29	0.084883106	0.104263738	-0.0193806
18	0E5A	0.148870357	0.086363727	0.06250663
19	0H11	0.167902214	0.076897549	0.09100467
20	0L10	0.091937447	0.103654839	-0.0117174
21	0L16	-0.162426343	0.109747837	0.05267851
22	0N25	0.134787281	0.078329675	0.05645761
23	0P15	0.216080775	0.046738833	0.16934194
24	0P4A	0.153782641	0.037182636	0.1166
25	0R28	-0.170326478	0.115363783	0.05496269
26	0R36	0.087421356	0.102628733	-0.0152074
27	0R43	-0.021365213	0.124363738	-0.1029985
28	0R7A	0.017904692	0.121242537	-0.1033378
29	0U70	0.05314702	0.038728737	0.01441828
30	0U73	0.136829126	0.097363526	0.0394656
31	0U78	0.049281773	0.056728283	-0.0074465
32	0U81	0.215432614	0.109736673	0.10569594
33	0X7A	0.105874322	0.125387329	-0.019513
34	C40A	-0.127856419	0.045272829	0.08258359
35	CBL1	-0.093752912	0.122367348	-0.0286144
36	CF48	0.145623988	0.034267385	0.1113566
37	CFA3	0.213369773	0.105363739	0.10800603
38	D12A	0.14237888	0.075346739	0.06703214
39	D35A	0.05094214	0.113463824	-0.0625217
40	K17A	0.089671909	0.103353437	-0.0136815
41	K38A	0.167044728	0.104637328	0.0624074
42	L18A	0.113317969	0.121829209	-0.0085112
43	L29A	0.148280958	0.125363829	0.02291713
44	MW60	-0.081825839	0.069812736	0.0120131
45	N102	0.183152398	0.103643264	0.07950913

46	N120	0.104820892	0.114353638	-0.0095327
47	N127	0.07934897	0.092537389	-0.0131884
48	N133	0.142157699	0.086729364	0.05542834
49	0R16	0.104996863	0.120325627	-0.0153288
50	XB92	0.104960488	0.132537389	-0.0275769
51	XV55	-0.073731635	0.022427122	0.05130451
52	ZVS3	-0.064207506	0.026354675	0.03785283

Approach 1 had a minimum residual error of -0.02m was seen at station 0R43 while the maximum residual error 0.23m was at station 00P6 in the North component while Approach 3 recorded a minimum value of -0.01m at station 0B2A and the maximum residual error of 0.13m at station XB92. Approach 1 had 38% of its station closer to the standard while Approach 3 had 62% of its station closer to the standard in the north component. Table 4.7 presents the absolute residual difference between approach 1 and approach 3 in the height component.

Table 0.7: Residual Difference between Approach 1 and Approach 3 in the Height Component

S/N	**Stations**	**APPROACH 1 HEIGHT**	**APPROACH 3 HEIGHT**	**APP1-APP3 (dH)**
1	00F7	0.138927399	0.067346734	0.07158066
2	00H4	0.224549287	0.167346787	0.0572025
3	00H5	-0.284746742	0.214648734	0.07009801
4	00L8	0.248489753	0.134325633	0.11416412
5	00P6	0.256743735	0.153267329	0.10347641
6	00X1	0.154648349	-0.05432532	0.10032303
7	00X3	0.127546735	-0.018738734	0.108808
8	00X4	-0.148738914	0.06838934	0.08034957
9	00X6	0.176356744	0.087474332	0.08888241
10	0A10	0.141848759	0.167327323	-0.0254786
11	0A16	0.197546755	-0.137874874	0.05967188
12	0A24	0.255347347	0.188987437	0.06635991

13	0A39	0.146348735	0.158734873	-0.0123861
14	0B2A	0.243734893	-0.073676732	0.17005816
15	0C16	0.389348934	0.195632563	0.19371637
16	0C32	0.343256327	0.083632563	0.25962376
17	0D29	0.009497347	0.117348735	-0.1078514
S/N	**Stations**	**EAST (m)**	**NORTH (m)**	**HEIGHT (m)**
18	0E5A	0.073635637	0.100673487	-0.0270379
19	0H11	0.235638735	0.138743343	0.09689539
20	0L10	0.218767347	-0.187347323	0.03142002
21	0L16	0.154346735	0.167348735	-0.013002
22	0N25	-0.132928935	0.16472329	-0.0317944
23	0P15	0.258747867	0.026734749	0.23201312
24	0P4A	0.321234123	0.167348749	0.15388537
25	0R28	0.187348755	0.038754874	0.14859388
26	0R36	0.107548755	0.126734267	-0.0191855
27	0R43	0.048938734	-0.138732873	-0.0897941
28	0R7A	0.154325433	0.032426731	0.1218987
29	0U70	0.102634669	0.132873489	-0.0302388
30	0U73	0.117328747	0.041256673	0.07607207
31	0U78	0.243587349	0.10543654	0.13815081
32	0U81	-0.051674873	0.085323987	-0.0336491
33	0X7A	0.141875488	0.167487438	-0.025612
34	C40A	0.156346735	0.167348743	-0.011002
35	CBL1	0.138734287	0.109348749	0.02938554
36	CF48	0.123434999	0.047328732	0.07610627
37	CFA3	0.072532634	0.11898934	-0.0464567
38	D12A	0.142428955	0.06832832	0.07410063
39	D35A	0.005378992	0.098732732	-0.0933537
40	K17A	0.056326737	0.03288587	0.02344087
41	K38A	-0.042423632	0.056342632	-0.013919
42	L18A	0.063673467	0.054893443	0.00878002
43	L29A	0.143448733	0.187348754	-0.0439
44	MW60	0.156325439	0.096734673	0.05959077
45	N102	-0.118734735	0.167347334	-0.0486126
46	N120	0.142425789	0.048732871	0.09369292
47	N127	0.196456342	-0.104254093	0.09220225
48	N133	0.274875489	0.056325633	0.21854986
49	0R16	-0.157438743	-0.016732673	0.14070607
50	XB92	0.123232732	0.132893487	-0.0096608

51	XV55	-0.175432563	0.156326327	0.01910624
52	ZVS3	0.151673473	0.14326735	0.00840612

For Approach 1, minimum residual error of -0.05m was seen at station 0U81 while the maximum residual error of 0.39m was encountered at station 0C16 while Approach 3 had a minimum residual error -0.02m at station 00X3 and a maximum value of 0.21m was seen at station 00H5. Approach 1 had 34% of its station closer to the standard values while Approach 3 had 65% of its station closer to the standard values in the height component. This indicates that Approach 3 which is the PPP approach gave an outstanding result than Approach 1 in both the East, North and Height components respectively.

Approach 1 gave reduced observational campaign observed from pairs of station combined to form a network. This processing strategy gave rise to a much smaller network and reduced inter-station distance between the reference stations and the unknown points. With shorter baseline common mode errors between the two points cancels each other and those that are spatially correlated between the two stations will be reduced giving rise to more accurate positioning as against Approach 2. This is also true for satellite orbital errors and clock errors. Furthermore, there is relative ease of resolving ambiguity to integer values. However the drawback of this technique is the dependency of good quality observables and free data gaps from the reference stations with all required specification been equal acceptable in differencing technique to realized high positioning accuracy.

However Approach 3 had the highest percentage of performance in all the three components. In other words solution from Precise Point Positioning (PPP) approach gave coordinate values very much closer to the standard values. This technique performed much better because of the rigorous modeling of errors and the quality of products obtained from

IGS used in the processing. This is so because it calculates position of the satellite data without reference to data supplied from another receiver or base station

This technique of finding precise coordinates of a point is simple and very economical as against to other approaches that is more costly and complex though robust. This implies that PPP technique can be used in many application requiring high accuracies even in a very isolated place where reference station like Continuous Operating System (CORS) is lacking or out of range to the area of survey. This approach is well suitable for use in Nigeria because with the establishment of the Permanent GNSS Network (NIGNET) since 2010 in Nigeria, the spatial coverage of the CORs on a distance of more than 500km (between the stations) seems inadequate based on the global station densification standard of the International Terrestrial Reference Frame (ITRF) (Ojigi, 2014). In addition, the CORS current architecture, post processing system of the CORS in Nigeria does not meet the multi-level application needs in the areas of cadastral and large scale mapping, mobile Geographic Information System (GIS), rapid mapping and positioning in oil and gas, precision agriculture, mineral prospecting, marine/oceanography and marine safety administration, flood and drainage control, engineering and construction surveys, civil aviation and navigation, road and rail transport, asset tracking, structural deformation and subsidence, infrastructure and location based systems, etc. (Ojigi, 2014; Ali and Samir, 2014) but the use of this technique is a suitable substitute since it positioning accuracy is comparable to that of relative positioning

Although PPP presents definite advantages, its applicability is currently limited by the long convergence time and the availability of quality accurate error corrections because when the errors are well represented and accounted gives Approach 3 high level of flexibility and usage.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Summary of Findings

i. The geometric analysis of the existing Nigerian Primary Network using Triangle Inequality theorem revealed that the network fulfilled the condition of the theorem however, when subjected to how well-conditioned the triangles were within the network, it was discovered that 56% of the triangles met the requirement while 44% did not meet the condition.

ii. Comparative analysis of all the processing strategies employed; Baseline solutions (Approach 1), Network solutions, (Approach 2) and Precise Positioning Point solutions (Approach3) with respect to standard coordinates value; Approach 1 had residual error that ranges from -0.02m to 0.24m, -0.02m to 0.23m and -0.05m to 0.39 in the East, North and Height component while Approach 2 had residual error between -0.15m to 0.93m, -0.05m to 0.98m, and -0.04m to 0.99m in the East, North, and Height but Approach 3 had its residual error between -0.02m to 0.15m, -0.01m to 0.13m, and -0.02m to 0.21 in the height components respectively.

iii. Approach 1 and Approach 3 provided the closest estimates of the coordinate solutions to the standard. However, comparison between the two approaches with respect to the points of reference (standard) had 42% of stations of Approach 1 best approximated to the standard coordinates and 58% of stations of the Approach 3 best approximated the coordinates of the standard points in the East component

iv. Approach 1 had 38% of the stations nearer to the standard coordinate solutions while Approach 3 had 62% of the stations had coordinate solution closer to the standard in the North component.

v. Approach 1 had 34% of its stations closer to the standard coordinate solutions whilst Approach 3 had 66% of its stations coordinate solution closer to the standard while in the North component. This implies that the coordinate outputs of the PPP solution are better than Approach 1 and Approach 2 solutions.

5.2 Conclusion

This study carried out 3D readjustment of part of the Nigerian Primary Triangulation Network using GNSS data obtained from Office of the Surveyor General of the Federation (OSGoF). The GNSS observational campaign was carried out within the period of October, 2010 – April, 2011. The processing and adjustment was done using fifty-two stations raw GNSS observational campaign data GNSS with different processing strategies referenced to ITRF2008. These processing strategies include; reducing observational campaign observed from pairs of stations (baselines) and combining these baselines into a network (Approach 1), taking GPS observations observed simultaneously at all stations directly into a network adjustment where all the coordinates of the network are presented as unknowns (Approach 2) and lastly, processing the observations using Precise Point Positioning techniques (Approach 3).

The first two approaches were processed using Trimble Total Control (TTC) while the Precise Point Positioning approach was processed using GNSS-Lab Tool (gLAB). Solution of these different approaches were generated and comparative evaluation of the approaches was carried out to ascertain which of the approaches gives the best approximation to the points of reference or otherwise referred as standard in the study. The difference between the approach solutions and standard which is the residual error was determined and Approach 1 had 42%, 38% and 34% of its stations nearer to the standard coordinate value in the East, North and Height component respectively while Approach 3 had 58%, 61% and 66% of its stations best approximated in the East, North and Height components respectively. Approach 1 and 2 gave the least performance due to some restraining factors that considerably induced errors within its solution. These factors includes effects of long baselines and lack of robust optimal guiding procedure or algorithm for users in selecting reference stations for optimal results to avoid the effects of station selection on coordinate solution of the unknown because different station selection results to different solution for the unknown.

PPP solution gave the best approximation with respect to the point of reference. PPP is very capable of delivering high accuracy point positioning solutions in post-processing mode. The minimal difference between the PPP solution and that of the points of reference were as a result of rigorous modeling of errors that can considerably degrades the accuracy of the point positions. The residual biases were typically at millimeter-centimeter level.

However, the PPP solution is limited to the quality of the correctional data used for the processing and it also requires time to converge to a solution and hence, not applicable to works that requires real-time application.

5.3 Recommendations

From the above results, analysis and conclusions of the study, the following are hereby recommended:

i. Densification of the First-order monument stations for the entire country is required to improve the effectiveness and efficiency of the Nigerian geodetic reference system using GNSS techniques where GNSS observations in Nigeria should be referenced to enable smooth integration of satellite data into the reference frame thereby, improving the quality of surveys and mapping activities in Nigeria. Furthermore, Government survey agencies e.g. OSGoF; who have the exclusive rights to the provision of these controls, should also seek for support and participation of the private sectors that have the requisite capability, using set out specifications.

ii. Different approaches gives different results (solutions) due to many factors. Therefore, there is need to adopt a standard approach for harmonizing GNSS solutions in the near future.

iii. Adequate rigorous planning should be attended to avoid defects in the design of GNSS campaign for the establishment of such controls round the country.

iv. The use of more rigorous robust scientific processing software such BERNESSE, GAMIT, etc. should be more employed as most commercial software's such as Leica Geo-Office, Trimble Total Control, Trimble Business Center are limited in providing very high accurate results for relative positioning solutions with baseline lengths of over 300km.

5.4 Research Contribution

i. The study identified and considered different approaches of point positioning using GNSS. Using different processing strategies with same dataset can give rise to different point solutions. The study has been able to compare these different solutions with respect to reference point solution to determine which approach gave high accurate result. Processed GNSS observations that gives a better approximation to the points of reference or known value are simply suitable for GNSS applications. The study was able to give credence to the accuracy capability and suitability of using Precise Point Positioning approach for geodetic applications requiring high positioning accuracy.

ii. The study has also been able to identify some pitfalls to avoid when combining simultaneous GNSS observations to yield ultimate result.

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APPENDICES

APPENDIX I: DETAILS OF THE NIGERIA TRIANGULATION NETWORK

Stations	LAT.	LONG.	Orthometric Height(m)	Distance (km)	ANGLES (°)	Station To
R43	12.02125	5.894593	456.44	157.4386	79.59525	R16
R16	12.69258	4.656074	335.118	130.0843	56.71921	R28
R28	13.57862	5.396466	348.972	185.1564	43.68554	R43
R43	12.02125	5.894593	456.44	185.1564	43.80453	R28
R28	13.57862	5.396466	348.972	105.1549	102.536	R36
R36	13.13372	6.224632	412.254	131.3217	33.65945	R43
R43	12.02125	5.894593	456.44	131.3217	46.75124	R36
R36	13.13372	6.224632	412.254	129.6484	67.50974	N127
N127	12.14852	6.816151	754.644	103.5925	65.73902	R43
N127	12.14852	6.816151	754.644	129.6484	33.51273	R36
R36	13.13372	6.224632	412.254	289.9146	21.48599	K001
K001	12.0143	8.566396	482.988	195.427	125.0013	N127
N133	11.87248	7.95495	618.726	130.557	9.373597	N127
N127	12.14852	6.816151	754.644	195.427	17.77548	K001

K001	12.0143	8.566396	482.988	69.92872	152.8509	N133
N133	11.87248	7.95495	618.726	69.92872	85.51077	K001
K001	12.0143	8.566396	482.988	254.0796	15.65646	N10
N10	9.785181	8.91557	1421.24	258.237	78.83277	N133
N10	9.785181	8.91557	1421.24	254.0796	50.47233	K001
K001	12.0143	8.566396	482.988	230.865	70.59435	A042
A042	10.96978	10.3495	585.132	207.8971	58.93332	N10

Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
A042	10.96978	10.3495	585.132	230.865	9.021411	K001
K001	12.0143	8.566396	482.988	221.7417	99.8473	A39
A39	11.28886	10.4174	474.746	36.75285	71.13129	A042
A39	11.28886	10.4174	474.746	221.7417	20.96199	K001
K001	12.0143	8.566396	482.988	566.865	12.50521	CFL60
CFL60	11.952	13.65824	0	368.4579	146.5328	A39
A24	10.6038	11.33967	605.629	117.6753	37.30428	A042
A042	10.96978	10.3495	585.132	384.6453	13.74869	CFL60
CFL60	11.952	13.65824	0	299.5616	128.947	A24

A021	10.45527	11.62795	740.331	36.18071	58.05608	A24
A24	10.6038	11.33967	605.629	299.5616	6.242913	CFL60
CFL60	11.952	13.65824	0	282.0666	115.701	A021
A16	10.12492	12.37588	768.768	91.1641	60.83355	A021
A021	10.45527	11.62795	740.331	282.0666	18.50941	CFL60
CFL60	11.952	13.65824	0	250.5418	100.657	A16
A16	10.12492	12.37588	768.768	239.6364	21.08092	E10
E10	9.190843	10.43992	312.6	193.9917	108.8989	A021
A021	10.45527	11.62795	740.331	91.1641	50.02023	A16
A16	10.12492	12.37588	768.768	283.5465	57.64568	C036
C036	7.998777	10.99304	524.471	146.8623	91.17907	E10
E10	9.190843	10.43992	312.6	239.6364	31.17525	A16
C036	7.998777	10.99304	524.471	100.7742	62.00416	C32
C32	7.758221	10.12051	373.029	164.0863	37.28768	E10
E10	9.190843	10.43992	312.6	146.8623	99.29185	C036
E10	9.190843	10.43992	312.6	187.6064	94.72633	A24
A24	10.6038	11.33967	605.629	36.18071	74.56659	A021
A021	10.45527	11.62795	740.331	193.9917	10.70708	E10

E10	9.190843	10.43992	312.6	200.092	66.59211	A042
A042	10.96978	10.3495	585.132	117.6753	78.27057	A24
A24	10.6038	11.33967	605.629	187.6064	35.13733	E10
E10	9.190843	10.43992	312.6	182.307	61.33205	N10
N10	9.785181	8.91557	1421.24	207.8971	53.01927	A042
A042	10.96978	10.3495	585.132	200.092	65.64868	E10
N10	9.785181	8.91557	1421.24	289.2695	116.9362	N120
N120	11.26529	6.790665	644.573	146.6194	93.44682	N133
N133	11.87248	7.95495	618.726	258.237	30.38304	N10
N120	11.26529	6.790665	644.573	99.80545	77.76473	N127
N127	12.14852	6.816151	754.644	130.557	41.76312	N133
N133	11.87248	7.95495	618.726	146.6194	60.47215	N120
C32	7.758221	10.12051	373.029	156.4321	50.58675	H11
H11	8.247428	8.803744	259.5107m	210.5139	47.37249	E10
E10	9.190843	10.43992	312.6	164.0863	82.04076	C32
N123	11.44118	6.991998	747.912	29.933	46.83431	N120
N120	11.26529	6.790665	644.573	99.80545	15.38486	N127
N127	12.14852	6.816151	754.644	82.2862	117.7808	N123

Stations	LAT.	LONG.	Orthometric Height(m)	Distance (km)	ANGLES (^{o})	Station To
N120	11.26529	6.790665	644.573	131.2936	48.59884	R43
R43	12.02125	5.894593	456.44	103.5925	80.35066	N127
N127	12.14852	6.816151	754.644	99.80545	51.05051	N120
R43	12.02125	5.894593	456.44	192.1162	43.12239	BK05
BK05	12.47491	4.231568	0	53.30692	123.5148	R16
R16	12.69258	4.656074	335.118	157.4386	13.3628	R43
BK05	12.47491	4.231568	0	164.7442	92.28908	D29
D29	11.38796	5.217308	505.549	101.7255	56.65988	D17
D17	10.76074	4.560171	325.526	196.9944	31.05103	BK05
D17	10.76074	4.560171	325.526	57.74363	23.14615	D013
D013	10.2756	4.390668	268.118	248.7733	6.616192	BK05
BK05	12.47491	4.231568	0	196.9944	150.2377	D17
D013	10.2756	4.390668	268.118	57.74363	79.50594	D17
D17	10.76074	4.560171	325.526	255.7027	13.03263	N102
N102	9.639053	6.558701	449.616	251.7061	87.46143	D013

D17	10.76074	4.560171	325.526	101.7255	83.14613	D29
D29	11.38796	5.217308	505.549	246.9886	23.25735	N102
N102	9.639053	6.558701	449.616	255.7027	73.59652	D17
N102	9.639053	6.558701	449.616	246.9886	48.2652	D29
D29	11.38796	5.217308	505.549	175.691	86.51879	N120
N120	11.26529	6.790665	644.573	184.7153	45.21601	N102
N120	11.26529	6.790665	644.573	175.691	48.06426	D29

Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
D29	11.38796	5.217308	505.549	103.9292	95.90947	R43
R43	12.02125	5.894593	456.44	131.2936	36.02627	N120
R43	12.02125	5.894593	456.44	103.9292	88.23533	D29
D29	11.38796	5.217308	505.549	164.7442	32.7198	BK05
BK05	12.47491	4.231568	0	192.1162	59.04488	R43
N102	9.639053	6.558701	449.616	184.7153	1.448503	N120
N120	11.26529	6.790665	644.573	185.7807	76.43419	L40
L40	9.636028	6.515648	281.13	4.804636	102.1173	N102
L40	9.636028	6.515648	281.13	185.7807	21.59136	N120

N120	11.26529	6.790665	644.573	1464.107	356.9697	N123
N123				1293.182	204.6217	L40
L40	9.636028	6.515648	281.13			
N120	11.26529	6.790665	644.573	246.9272	36.95019	N032
N032	9.105985	7.20157	686.068	64.33634	131.862	N107
N107	9.492796	6.77451	543.706	199.313	11.18782	N120
N120	11.26529	6.790665	644.573	292.7252	53.49062	N25
N25	8.998776	8.087372	570.12	99.33525	107.6464	N032
N032	9.105985	7.20157	686.068	246.9272	18.86294	N120
N120	11.26529	6.790665	644.573	289.2695	78.8994	N10
N10	9.785181	8.91557	1421.24	127.5578	75.79879	N25
N25	8.998776	8.087372	570.12	292.7252	25.30181	N120
N10	9.785181	8.91557	1421.24	172.6464	47.61296	H11
Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
H11	8.247428	8.803744	259.5107m	115.8562	90.25539	N25
N25	8.998776	8.087372	570.12	127.5578	42.13165	N10
N10	9.785181	8.91557	1421.24	182.307	51.58674	E10

E10	9.190843	10.43992	312.6	210.5139	55.74918	H11
H11	8.247428	8.803744	259.5107m	172.6464	72.66408	N10
E10	9.190843	10.43992	312.6	164.0863	82.04076	C32
C32	7.758221	10.12051	373.029	156.4321	50.58675	H11
H11	8.247428	8.803744	259.5107m	210.5139	47.37249	E10
C036	7.998777	10.99304	524.471	301.5917	12.54278	C16
C16	6.137064	9.026654	607.473	217.8227	139.4605	C32
C32	7.758221	10.12051	373.029	100.7742	27.99668	C036
C32	7.758221	10.12051	373.029	217.8227	51.67457	C16
C16	6.137064	9.026654	607.473	154.9825	83.37629	H004
H004	7.462727	8.603204	341.778	172.0934	44.94915	C32
H11	8.247428	8.803744	259.5107m	84.24793	84.75148	H5
H5	7.511766	8.972309	300.431	41.45109	68.09322	H004
H004	7.462727	8.603204	341.778	90.40107	27.1553	H11
C32	7.758221	10.12051	373.029	130.7419	90.69139	H5
H5	7.511766	8.972309	300.431	84.24793	56.66666	H11
H11	8.247428	8.803744	259.5107m	156.4321	32.64195	C32

C32	7.758221	10.12051	373.029	217.8227	51.67457	C16
C16	6.137064	9.026654	607.473	154.9825	83.37629	H004

Stations	LAT.	LONG.	Orthometric Height(m)	Distance (km)	ANGLES (^{o})	Station To
H004	7.462727	8.603204	341.778	172.0934	44.94915	C32
C16	6.137064	9.026654	607.473	45.38282	100.2548	C14
C14	6.202751	8.624291	127.232	140.3534	16.74211	H004
H004	7.462727	8.603204	341.778	154.9825	63.00312	C16
C14	6.202751	8.624291	127.232	140.3534	16.74211	H004
H004	7.462727	8.603204	341.778	154.9825	63.00312	C16
C16	6.137064	9.026654	607.473	45.38282	100.2548	C14
C14	6.202751	8.624291	127.232	45.38282	44.6459	C16
C16	6.137064	9.026654	607.473	123.3166	19.28874	C008
C008	5.495553	8.122862	279.5668	96.41456	116.0654	C14
C008	5.495553	8.122862	279.5668	123.3166	20.49358	C16
C16	6.137064	9.026654	607.473	136.3235	64.24003	MW606
MW606	5.122032	8.338829	90.472	47.94537	95.26638	C008

C14	6.202751	8.624291	127.232	123.3039	51.09138	U013
U013	6.264034	7.518327	299.085	180.0322	43.07083	H004
H004	7.462727	8.603204	341.778	140.3534	85.83779	C14
MW606	5.122032	8.338829	90.472	146.8991	19.03927	ZVS3003
ZVS3003	4.847971	7.047818	16.614	139.6128	89.06037	C008
C008	5.495553	8.122862	279.5668	47.94537	71.90037	MW606

Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
ZVS3003	4.847971	7.047818	16.614	165.8224	56.61878	U013
U013	6.264034	7.518327	299.085	108.7527	82.80891	C008
C008	5.495553	8.122862	279.5668	139.6128	40.57231	ZVS3003
U013	6.264034	7.518327	299.085	165.8224	31.08548	ZVS3003
ZVS3003	4.847971	7.047818	16.614	150.9063	84.09614	CFH66

CFH66	6.173144	6.750133	37.214	86.11056	64.81838	U013
ZVS3003	4.847971	7.047818	16.614	164.7881	59.8304	CBL10
CBL10	5.539285	5.737915	4.793	132.8966	70.63795	CFH66
CFH66	6.173144	6.750133	37.214	150.9063	49.53165	ZVS3003
CBL10	5.539285	5.737915	4.793	161.8316	54.24422	U081
U081	6.787012	6.485677	193.72	74.39334	98.74049	CFH66
CFH66	6.173144	6.750133	37.214	132.8966	27.0153	CBL10
U013	6.264034	7.518327	299.085	86.11056	42.84418	CFH66
CFH66	6.173144	6.750133	37.214	88.15901	66.82709	U78
U78	6.776416	7.263365	402.003	63.70243	70.32873	U013
U013	6.264034	7.518327	299.085	63.70243	90.72718	U78
U78	6.776416	7.263365	402.003	167.6184	20.7124	H004
Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
H004	7.462727	8.603204	341.778	180.0322	68.56042	U013

U081	6.787012	6.485677	193.72	86.57987	50.43723	U78
U78	6.776416	7.263365	402.003	88.15901	63.68124	CFH66
CFH66	6.173144	6.750133	37.214	74.39334	65.88153	U081
U78	6.776416	7.263365	402.003	86.57987	78.28351	U081
U081	6.787012	6.485677	193.72	116.5698	40.57438	U70
U70	7.807632	6.713009	411.485	130.2746	61.1421	U78
U78	6.776416	7.263365	402.003	130.2746	51.56247	U70
U70	7.807632	6.713009	411.485	213.9058	37.57113	H004
H004	7.462727	8.603204	341.778	167.6184	89.13361	U78
U70	7.807632	6.713009	411.485	202.827	67.43477	N025
N025	8.998776	8.087372	570.12	180.9946	61.11375	H004
H004	7.462727	8.603204	341.778	213.9058	51.45147	U70
H004	7.462727	8.603204	341.778	180.9946	24.99001	N025
N025	8.998776	8.087372	570.12	115.8562	122.2292	H11
H11	8.247428	8.803744	259.5107m	90.40107	32.78083	H004
N025	8.998776	8.087372	570.12	202.827	28.41492	U70
U70	7.807632	6.713009	411.485	155.0203	103.5547	N032
N032	9.105985	7.20157	686.068	99.33525	48.03042	N025

U70	7.807632	6.713009	411.485	94.18006	46.05393	U73
U73	7.843734	5.86775	651.321	204.856	25.90368	H032
Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
H032	9.105985	7.20157	686.068	155.0203	108.0424	U70
U73	7.843734	5.86775	651.321	210.2946	76.2947	N107
N107	9.492796	6.77451	543.706	64.33634	85.94501	N032
N032	9.105985	7.20157	686.068	204.856	17.76029	U73
U73	7.843734	5.86775	651.321	207.4614	75.22227	L041
L041	9.585634	6.506468	234.356	94.22076	78.37878	N032
N032	9.105985	7.20157	686.068	204.856	26.39895	U73
CBL10	5.539285	5.737915	4.793	213.5916	46.60502	U072
U072	7.453668	5.870887	633.619	101.0132	106.4247	U081
U081	6.787012	6.485677	193.72	161.8316	26.97025	CBL10
U081	6.787012	6.485677	193.72	101.0132	70.23955	U072
U072	7.453668	5.870887	633.619	101.7242	54.59444	U70
U70	7.807632	6.713009	411.485	116.5698	55.16601	U081

U072	7.453668	5.870887	633.619	43.52511	87.05282	U73
U73	7.843734	5.86775	651.321	94.18006	25.35993	U70
U70	7.807632	6.713009	411.485	101.7242	67.58725	U072
U72	7.453668	5.870887	633.619	213.5916	33.38057	CBL10
CBL10	5.539285	5.737915	4.793	302.3309	43.45474	L16
L16	7.904115	4.403847	498.209	170.8705	103.1647	U72
U72	7.453668	5.870887	633.619	170.8705	14.73301	L16
L16	7.904115	4.403847	498.209	163.1003	92.8635	U73

Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES $(^{o})$**	**Station To**
U73	7.843734	5.86775	651.321	43.52511	72.40349	U72
L16	7.904115	4.403847	498.209	302.3309	323.6605	CBL10
CBL10	5.539285	5.737915	4.793	294.7719	74.07359	CFA
CFA	6.626905	3.323127	47.093	186.4073	69.58692	L16
CFA	6.626905	3.323127	47.093	90.77452	132.9804	L3
L3	7.41728	3.520866	264.284	112.3053	20.86899	L16
L16	7.904115	4.403847	498.209	186.4073	26.15057	CFA

L16	7.904115	4.403847	498.209	112.3053	158.4695	L3
L3	7.41728	3.520866	264.284	30.77358	16.94882	L10
L10	7.204402	3.344903	172.987	141.382	4.581661	L16
L10	7.204402	3.344903	172.987	89.54172	106.6204	L8
L8	7.956157	3.626676	439.411	86.7092	37.41122	L16
L16	7.904115	4.403847	498.209	141.382	35.96833	L10
L16	7.904115	4.403847	498.209	86.7092	35.94395	L8
L8	7.956157	3.626676	439.411	122.313	44.29401	L018
L018	8.536893	4.558033	404.216	72.75888	99.76205	L16
L16	7.904115	4.403847	498.209	72.75888	75.63737	L018
L018	8.536893	4.558033	404.216	165.09	25.59791	U73
U73	7.843734	5.86775	651.321	163.1003	78.76472	L16
U73	7.843734	5.86775	651.321	165.09	56.46216	L018
L018	8.536893	4.558033	404.216	246.6571	41.50495	L041
L041	9.585634	6.506468	234.356	207.4614	82.03289	U73

Stations	**LAT.**	**LONG.**	**Orthometric Height(m)**	**Distance (km)**	**ANGLES (^{o})**	**Station To**
L018	8.536893	4.558033	404.216	69.27188	128.1758	D06

D06	9.10883	4.795946	296.528	197.7717	12.75317	L41
L41	9.585634	6.506468	234.356	246.6571	39.07102	L018
L8	7.956157	3.626676	439.411	273.3789	23.57576	D013
D013	10.2756	4.390668	268.118	195.8072	116.609	L018
L018	8.536893	4.558033	404.216	122.313	39.81524	L8
L018	8.536893	4.558033	404.216	195.8072	13.55008	D013
D013	10.2756	4.390668	268.118	138.4656	138.5235	D06
D06	9.10883	4.795946	296.528	69.27188	27.92645	L018
D06	9.10883	4.795946	296.528	138.4656	54.07479	D013
D013	10.2756	4.390668	268.118	247.2128	34.10149	L40
L40	9.636028	6.515648	281.13	200.3496	91.82372	D06

APPENDIX II: TRIANGLE INEQUALITY

S/N	Stations from	ANGLES in the triangles	Distance in Kilometres	Station to	REMARK
	R43	79.59525	157.4386	R16	
1	R16	56.71921	130.0843	R28	Good
	R28	43.68554	185.1564	R43	
2	R43	43.80453	185.1564	R28	
	R28	102.536	105.1549	R36	Good
	R36	33.65945	131.3217	R43	
	R43	46.75124	131.3217	R36	
3	R36	67.50974	129.6484	N127	Good
	N127	65.73902	103.5925	R43	
4	N127	33.51273	129.6484	R36	
	R36	21.48599	289.9146	K001	Good
	K001	125.0013	195.427	N127	
	N133	9.373597	130.557	N127	
5	N127	17.77548	195.427	K001	Good
	K001	152.8509	69.92872	N133	

	N133	85.51077	69.92872	K001	
6	K001	15.65646	254.0796	N10	Good
	N10	78.83277	258.237	N133	
	N10	50.47233	254.0796	K001	
7	K001	70.59435	230.865	A042	Good
	A042	58.93332	207.8971	N10	
	A042	9.021411	230.865	K001	
8	K001	99.8473	221.7417	A39	Good
	A39	71.13129	36.75285	A024	
	A39	20.96199	221.7417	K001	
9	K001	12.50521	566.865	CFL60	Good
S/N	**Stations from**	**ANGLES in the triangles**	**Distance in Kilometres**	**Station to**	**REMARK**
	CFL60	146.5328	368.4579	A39	
10	A24	37.30428	117.6753	A042	
	A042	13.74869	384.6453	CFL60	Good
	CFL60	128.947	299.5616	A24	
11	A021	58.05608	36.18071	A24	
	A24	6.242913	299.5616	CFL60	Bad
	CFL60	115.701	282.0666	A021	
	A16	60.83355	91.1641	A021	

12	A021	18.50941	282.0666	CFL60	Bad
	CFL60	100.657	250.5418	A16	
	A16	21.08092	239.6364	E10	
13	E10	108.8989	193.9917	A021	Bad
	A021	50.02023	91.1641	A16	
	A16	57.64568	283.5465	C036	Good
14	C036	91.17907	146.8623	E10	
	E10	31.17525	239.6364	A16	
	C036	62.00416	100.7742	C32	
15	C32	37.28768	164.0863	E10	Good
	E10	99.29185	146.8623	C036	
	E10	94.72633	187.6064	A24	
16	A24	74.56659	36.18071	A021	Bad
	A021	10.70708	193.9917	E10	
	E10	66.59211	200.092	A042	
17	A042	78.27057	117.6753	A24	Good
	A24	35.13733	187.6064	E10	
	E10	61.33205	182.307	N10	
18	N10	53.01927	207.8971	A024	Good
S/N	**Stations from**	**ANGLES in the triangles**	**Distance in Kilometres**	**Station to**	**REMARK**
	A042	65.64868	200.092	E10	

	N10	116.9362	289.2695	N120	
19	N120	93.44682	146.6194	N133	Good
	N133	30.38304	258.237	N10	
	N120	77.76473	99.80545	N127	
20	N127	41.76312	130.557	N133	Good
	N133	60.47215	146.6194	N120	
	C32	50.58675	156.4321	H11	
21	H11	47.37249	210.5139	E10	Good
	E10	82.04076	164.0863	C32	
	N123	46.83431	29.933	120	
22	N120	15.38486	99.80545	127	Bad
	N127	117.7808	82.2862	N123	
	N120	48.59884	131.2936	R43	
23	R43	80.35066	103.5925	N127	
	N127	51.05051	99.80545	N120	Good
	R43	43.12239	192.1162	BK05	
24	BK05	123.5148	53.30692	R16	Good
	R16	13.3628	157.4386	R43	
	BK05	92.28908	164.7442	D29	
25	D29	56.65988	101.7255	D17	Good
	D17	31.05103	196.9944	BK05	
	D17	23.14615	57.74363	D013	

S/N	Stations from	ANGLES in the triangles	Distance in Kilometres	Station to	REMARK
26	D013	6.616192	248.7733	BK05	Bad
	BK05	150.2377	196.9944	D17	
	D013	79.50594	57.74363	D17	
27	D17	13.03263	255.7027	N102	Bad
	N102	87.46143	251.7061	D013	
	D17	83.14613	101.7255	D29	
28	D29	23.25735	246.9886	N102	Bad
	N102	73.59652	255.7027	D17	
	N102	48.2652	246.9886	D29	
29	D29	86.51879	175.691	N120	Good
	N120	45.21601	184.7153	N102	
	N120	48.06426	175.691	D29	
30	D29	95.90947	103.9292	R43	Good
	R43	36.02627	131.2936	N120	
	R43	88.23533	103.9292	D29	
31	D29	32.7198	164.7442	BK05	Good
	BK05	59.04488	192.1162	R43	
	N102	1.448503	184.7153	N120	
32	N120	76.43419	185.7807	L40	Good

	L40	102.1173	4.804636	N102	
	L40	21.59136	185.7807	N120	
33	N120	356.9697	1464.107	N123	Bad
	N123	204.6217	1293.182	L40	
	N120	36.95019	246.9272	N032	
34	N032	131.862	64.33634	N107	Bad
	N107	11.18782	199.313	N120	
	N120	53.49062	292.7252	N25	
35	N25	107.6464	99.33525	N032	Bad
	N032	18.86294	246.9272	N120	
	N120	78.8994	289.2695	N10	

S/N	**Stations from**	**ANGLES in the triangles**	**Distance in Kilometres**	**Station to**	**REMARK**
36	N10	75.79879	127.5578	N25	Bad
	N25	25.30181	292.7252	N120	
	N10	47.61296	172.6464	H11	
37	H11	90.25539	115.8562	N25	Good
	N25	42.13165	127.5578	N10	
	N10	51.58674	182.307	E10	
38	E10	55.74918	210.5139	H11	Good

	H11	72.66408	172.6464	N10	
	E10	82.04076	164.0863	C32	
39	C32	50.58675	156.4321	H11	Good
	H11	47.37249	210.5139	E10	
	C036	12.54278	301.5917	C16	
40	C16	139.4605	217.8227	C32	Bad
	C32	27.99668	100.7742	C036	
	C32	51.67457	217.8227	C16	
41	C16	83.37629	154.9825	H004	Good
	H004	44.94915	172.0934	C32	
	H11	84.75148	84.24793	H5	
42	H5	68.09322	41.45109	H004	Bad
	H004	27.1553	90.40107	H11	
	C32	90.69139	130.7419	H5	
43	H5	56.66666	84.24793	H11	Good
	H11	32.64195	156.4321	C32	
	C32	51.67457	217.8227	C16	
44	C16	83.37629	154.9825	H004	Good
	H004	44.94915	172.0934	C32	
	C16	100.2548	45.38282	C14	
S/N	**Stations from**	**ANGLES in the triangles**	**Distance in Kilometres**	**Station to**	**REMARK**

45	C14	16.74211	140.3534	C16	Bad
	H004	63.00312	154.9825	H004	
	C14	16.74211	140.3534	H004	
46	H004	63.00312	154.9825	C16	Bad
	C16	100.2548	45.38282	C14	
	C14	44.6459	45.38282	C16	
47	C16	19.28874	123.3166	C008	Bad
	C008	116.0654	96.41456	C14	
	C008	20.49358	123.3166	C16	
48	C16	64.24003	136.3235	MW606	Bad
	MW606	95.26638	47.94537	C008	
	C14	51.09138	123.3039	U013	
49	U013	43.07083	180.0322	H004	Good
	H004	85.83779	140.3534	C14	
	MW606	19.03927	146.8991	ZVS3003	
50	ZVS3003	89.06037	139.6128	C008	
	C008	71.90037	47.94537	MW606	Bad
	ZVS3003	56.61878	165.8224	U013	
51	U013	82.80891	108.7527	C008	Good
	C008	40.57231	139.6128	ZVS3003	
	U013	31.08548	165.8224	ZVS3003	

52	ZVS3003	84.09614	150.9063	CFH66	Good
	CFH66	64.81838	86.11056	U013	
	ZVS3003	59.8304	164.7881	CBL10	
53	CBL10	70.63795	132.8966	CFH66	Good

S/N	Stations from	ANGLES in the triangles	Distance in Kilometres	Station to	REMARK
	CFH66	49.53165	150.9063	ZVS3003	
	CBL10	54.24422	161.8316	U081	
54	U081	98.74049	74.39334	CFH66	Bad
	CFH66	27.0153	132.8966	CBL10	
	U013	42.84418	86.11056	CFH66	
55	CFH66	66.82709	88.15901	U78	Good
	U78	70.32873	63.70243	U013	
	U013	90.72718	63.70243	U78	
56	U78	20.7124	167.6184	H004	Bad
	H004	68.56042	180.0322	U013	
	U081	50.43723	86.57987	U78	
57	U78	63.68124	88.15901	CFH66	Good
	CFH66	65.88153	74.39334	U081	

	U78	78.28351	86.57987	U081	
58	U081	40.57438	116.5698	U70	Good
	U70	61.1421	130.2746	U78	
	U78	51.56247	130.2746	U70	
59	U70	37.57113	213.9058	H004	Good
	H004	89.13361	167.6184	U78	
	U70	67.43477	202.827	N025	
60	N025	61.11375	180.9946	H004	Good
	H004	51.45147	213.9058	U70	
	H004	24.99001	180.9946	N025	
61	N025	122.2292	115.8562	H11	Bad
	H11	32.78083	90.40107	H004	
	N025	28.41492	202.827	U70	

S/N	**Stations from**	**ANGLES in the triangles**	**Distance in Kilometres**	**Station to**	**REMARK**
62	U70	103.5547	155.0203	N032	Bad
	N032	48.03042	99.33525	N025	
	U70	46.05393	94.18006	U73	
63	U73	25.90368	204.856	H032	Bad
	H032	108.0424	155.0203	U70	
	U73	76.2947	210.2946	N107	
64	N107	85.94501	64.33634	N032	Bad

S/N	Stations from	ANGLES in the triangles	Distance in Kilometres	Station to	REMARK
	N032	17.76029	204.856	U73	
	U73	75.22227	207.4614	L041	
65	L041	78.37878	94.22076	N032	
	N032	26.39895	204.856	U73	Bad
	CBL10	46.60502	213.5916	U072	
67	U072	106.4247	101.0132	U081	Bad
	U081	26.97025	161.8316	CBL10	
	U081	70.23955	101.0132	U072	
68	U072	54.59444	101.7242	U70	Good
	U70	55.16601	116.5698	U081	
	U072	87.05282	43.52511	U73	
69	U73	25.35993	94.18006	U70	Good
	U70	67.58725	101.7242	U072	
	U72	33.38057	213.5916	CBL10	
70	CBL10	43.45474	302.3309	L16	Good
	L16	103.1647	170.8705	U72	
	U72	14.73301	170.8705	L16	
71	L16	92.8635	163.1003	U73	Bad
	U73	72.40349	43.52511	U72	
	L16	323.6605	302.3309	CBL10	

72	CBL10	74.07359	294.7719	CFA	Bad
	CFA	69.58692	186.4073	L16	
	CFA	132.9804	90.77452	L3	
73	L3	20.86899	112.3053	L16	Bad
	L16	26.15057	186.4073	CFA	
	L16	158.4695	112.3053	L3	
74	L3	16.94882	30.77358	L10	Bad
	L10	4.581661	141.382	L16	
	L10	106.6204	89.54172	L8	
75	L8	37.41122	86.7092	L16	Good
	L16	35.96833	141.382	L10	
	L16	35.94395	86.7092	L8	
76	L8	44.29401	122.313	L018	Good
	L018	99.76205	72.75888	L16	
	L16	75.63737	72.75888	L018	
77	L018	25.59791	165.09	U73	Bad
	U73	78.76472	163.1003	L16	
	U73	56.46216	165.09	L018	
78	L018	41.50495	246.6571	L041	Good
	L041	82.03289	207.4614	U73	
	L018	128.1758	69.27188	D06	

79	D06	12.75317	197.7717	L41	Bad
	L41	39.07102	246.6571	L018	
	L8	23.57576	273.3789	D013	
80	D013	116.609	195.8072	L018	Bad
	L018	39.81524	122.313	L8	

S/N	Stations from	ANGLES in the triangles	Distance in Kilometres	Station to	REMARK
	L018				
		13.55008	195.8072	D013	
	D013				Bad
81		138.5235	138.4656	D06	
	D06	27.92645	69.27188	L018	
	D06	54.07479	138.4656	D013	
82	D013	34.10149	247.2128	L40	Good
	L40	91.82372	200.3496	D06	