Addis Ababa University

Addis Ababa Institute of Technology

School of Civil and Environmental Engineering

Msc in Geomatics and Geodesy

(Specialization in Geomatics)

Horizontal Positional Accuracy Assessment of an Orthopohoto: A case of Addis Ababa City

By:
Sisay walelign

Supervisor: Dr. Tulu Besha Bedada

A thesis submitted to the School of Graduate School of Addis Ababa University in the partial fulfillment of the requirements for the Degree of Master of Science in Geodesy and Geomatics specialized by Geomatics

June, 2019

ADDIS ABABA, ETHIOPIA

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

DEPARTMENT OF CIVIL AND ENVIRMONATLA ENGINEERING

Horizontal Positional Accuracy Assessment of an Orthopohoto: A case of Addis Ababa city

By:
Sisay walelign

Approved by board of examiners:

Advisor	Signature	Date
Internal Examiner	Signature	Date
External Examiner	Signature	Date
Chairman	Signature	Date

DECLARATION

I, the undersigned, declares that this thesis work is my original work carried out under supervision of Dr. Tulu Besha Bedada (PHD). It has not been presented for a degree in any universities and all sources of materials used for the thesis work have been properly acknowledged.

____________________ ____________________ ____________

Name Signature Date

Place: Addis Ababa

This thesis has been submitted for examination with my approval as a university advisor.

____________________ ____________________ ____________

Name Signature Date

Acknowledgement

I would like to sincerely thank all people who have contributed to this work. First and foremost I wish to acknowledge and thank my supervisor Dr. Tulu Besha Bedada, for his guidance and mentor throughout the study period.

His wealth of experience and rich knowledge, enthusiastic support, inspiration, personal guidance and unreserved help during my difficult moments has been of great value for me. I owe my sincere gratitude to Mr. Andinet Ashagery for his expert advice, wide knowledge and constructive comments and helping me data downloading for precise ephemeris. He spared no effort to transfer his vast knowledge and experience to me, as well as helping me to develop the ideas that came up to my mind in tackling the challenges as they arose. I am also deeply grateful to Mr. Zelalem Abate and Mr. Zerihun Amedemariam for their support in the course of the research.

I would also like to thank staff within the land holding adjudication project office in providing surveying equipment's for the field data collection as well as for their valuable comments on the research design. My special thanks goes to Mr. Beweket, without his support I couldn't get instruments for data collection, so thank you.

I was also extraordinarily fortunate to have very good friends during the undertaking my masters, in particular Mr Simegnew Jember and Mr. Robel zekarias during data collection.

My deepest gratitude goes to my family for their unflagging love and support throughout my life; this dissertation is simply impossible without them. Their love provided my inspiration and was my driving force. I owe them everything and wish I could show them just how much I love and appreciate them.

And last but certainly not the least thanks goes to almighty GOD that gives me everything in my life and to Bethel, my lovely wife, for her understanding, great patience, support and encouragement, and being the source of inspiration to keep me going.

To everyone mentioned here, my sincere thanks.

Abstract

This study tries to examine horizontal positional accuracy of orthophotoof Addis Ababa city using in-situ Global Positioning System (GPS) data. GPS data is processed using different processing systems like spectrum survey, Leica GeoOfice(LGO) and also online processing systems AUSPOS and OPUS in a least square sense with a tie to local and regional GPS reference stations to predict horizontal coordinates at twenty checkpoints to assess the accuracy of orthophoto. The accuracy of horizontal coordinates of the checkpoints derived from orthophoto was evaluated by comparing it with the corresponding coordinates measured directly from in-situ GPS data. When we compared the orthophoto with that of online processing AUSPS it is in a good agreement with GPS data observed at twenty checkpoints with a root mean square error of 9.3 cm in x- and 9.9 cm in y-coordinates. On the other hand, the horizontal accuracy of orthophoto is determined at 3.19 cm as it compared with twenty check points with 95% confidence level. Whereas, when the GPS data are processed using sokkia spectrum with a tie to local GPS reference station the horizontal coordinates of the orthophoto are in agreement with in-situ GPS coordinates at an accuracy of 7.2 cm and 21.39 cm in x and y-directions respectively, The finding of the result illustrates that the horizontal accuracy of orthophoto is determined at 8.81 cm with 95% confidence level, while the GPS data is processed with a tie to local GPS network. Therefore, it can be concluded that, the accuracy of the orthophoto is within the national standard of error budget (~ 25cm).

Key Words: AUSPOS, OPUS**,** orthophoto, GPS data, horizontal accuracy,

Table of Content

List of Tables

List of Figures

List of Abbreviations

ASPRS	American Society of Photogrammetry and Remote Sensing
AUSPOS	Australia's on-line static GPS positioning service
CORS	Continuously Operating Reference Station
EMA	Ethiopia Mapping Agency
GCP	Ground control point
GIS	Geographic information system
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
GSD	Ground sample distance
IGS	International GNSS Service
NSSDA	National Standard for Spatial Data Accuracy
OPUS	Online Positioning User Service
RINEX	Receiver Independent Exchange
RMS	Root Mean Square
RMSE	Root Mean Square Error
RTK	Real Time Kinematic
UTM	Universal Transverse Mercator
WGS	World Geodetic System

CHAPTER ONE

1. INTRODUCTION

1.1 Background of the Study

Orthophotos as a data source for compiling parcel data offer several benefits. An orthophoto offers continuous aerial coverage that includes all the existing features in the real world that are larger than the given pixel resolution. Orthophotos can be, and usually are, georeferenced. Orthophotos are also a historical record from which spatial and temporal changes can be derived. Advances in computer technology have made digital orthophotos more readily available. Improvements in computer processing, storage, and software have made them an increasingly practical choice. Although orthophotos generally offer significant benefits, not all orthophotos are created equal. In general, their accuracy and quality will vary based on the data used to process them, as well as on the particular production procedure.

Contributing error factors include the characteristics and calibration of equipment used for image capture such as the camera and or scanner. Additional elements affecting the rectification process include ground control points (accuracy, amount, and distribution), the aerial triangulation process, digital elevation models (DEM) (including the elevations of features above the DEM surface), and the method software used for rectification. Therefore, it is essential to note that orthophotos should be evaluated for spatial accuracy and inspected for image quality before acceptance or use. The spatial accuracy evaluation of the orthophoto and its suitability for a given application can be performed in a number of different ways.

One way to do this is to overlay it onto another reference (e.g., a vector-based graphic) information layer known to have higher accuracy. Differences or errors in feature locations between the orthophoto and reference layer are observed and quantified. The results of that error quantification are used to determine the accuracy of the orthophoto. Another way to test the spatial accuracy is by using the Global Positioning System (GPS) to determine accurate positions of features that can be easily identified on the orthophoto. Comparing the GPS coordinate values of these points to those measured on the orthophoto yields the sought after spatial accuracy evaluation. A guideline for such an accuracy assessment is provided by the National Standard for Spatial Data Accuracy[1].

[1] NSSDA (FGDC, 1998)

In addition to a spatial accuracy examination, orthophotos should undergo an image quality review as well. Image quality can deteriorate during image acquisition, image processing, rectification, and mosaicking. Aspects of image quality include brightness and contrast changes between images, recording of scratches and dirt as images, discontinuities at the edges of mosaicked image patches, and so forth. If some of these defects are not addressed, some local anomalies of inaccurate spatial positioning of features or systematic spatial errors may occur. In addition, image defects can result in incorrect interpretation of the real-world landscape

Accuracy of data sets acquired from aerial photogrammetric surveying and GPS measurements are dependent on a number of parameters that limit their measurement quality. For instance: multipath, the inherent satellite signal accuracy, signal transmission delay, receiver hardware and software limitations, satellite signal obstruction are some of the problems associated with GPS measurement. And accuracy of aerial photographs depends the quality of ground control points, exterior orientation parameters, onboard GPS corrections accuracy, aerial triangulation or block adjustment computations, etc. However, photogrammetric data are affected by different factors such as topographic variations, canopy, earth's curvature and other error sources introduced by computational procedures such as rectification and digitization. The rectification process is affected by the accuracy and spatial distribution of the ground control points, the aerial triangulation process, and digital elevation used for rectification. These errors reduce the spatial accuracy of photogrammetric data and cause geometric distortion and misorientation of linear features. orthorectification process reduces geometric errors inherent within photography and imagery[2]. The variables contributing to geometric errors include camera and sensor orientation, systematic error associated with camera and topographic relief displacement, and earth curvature. Greenfeld (2001) also explained that orthophotos generally offer significant benefits, but all orthophotos are not generated with equal accuracy. In general, the accuracy and quality of orthophoto varies based on the accuracy of the source data. Other contributing error factors include photo resolution, camera calibration, angels between photos or points and objects that appear only on photographs with very low subtended angles and other factors affect the accuracy of photogrammetry.

The accuracy of orthophoto and cadastral data has to be eavalutaed using in-situ ground measurements to realize the production of large scale maps (ranging from 1:500 to 1:10,000) based on orthophoto and digital line map that can be reliably used for practical cadastral applications. In

[2] Lawali and Waziri (2014)

recent years, much technological advancement has been made in the field of photogrammetry and the process of making maps using orthophotos. Because of this the quality of photogrammetric data can be examined by assessing the accuracy of vertical and horizontal coordinates of land parcel boundaries, point features and orientation of linear features. The positional accuracy achievable using photogrametric methods and the scale of the final map are determined based on mapping requirements and practice[3]. Spatial accuracy of photgrammetric data can be determined by using the Global Positioning System (GPS)[4]. The accuracy in the positions of photogrammetric datasets needs to be evaluated against in-situ ground-truth datasets using positional accuracy assessment technique. Positional accuracy assessment refers to the task of evaluating the absolute relative positions of the spatial objects on aerial photo and digital line map with respect to their true locations on the surface of the Earth.

Horizontal positional accuracy is a guess of the accuracy of the horizontal positions of the spatial objects that can be measured in terms of latitude and longitude or local easting and northing coordinates[5]. Whereas, the vertical positional accuracy refers to the uncertainty with which the vertical coordinates (ellipsoidal height/geocentric radius, orthometric height, levelled height) of spatial objects can be measured. In general, the positional accuracy of orthophoto refers to the accuracy at which the position coordinates (latitude, longitude and ellipsoidal height) of the spatial objects that are well recognized on the orthophoto are estimated in reference to their corresponding ground-truth coordinates acquired at the same locations using an independent ground based in-situ measurement such as Global Navigation Satellite System (GNSS) observations (Congalton and Green, 2009; American Society of Photogrammetry and Remote sensing[6]. In theory, the accuracy of ground based reference data used to validate the accuracy of photogrammetric data shall be at least three times more accurate than the photogrammetric data being assessed (Congalton and Green, 2009), while error in positional accuracy of photogrammetric data is the difference between the coordinates of the same selected spatial objects as acquired from photogrammetric survey when compared to coordinates measured using GNSS receivers. Most of the time positional accuracy assessment is done in point data but linear features also must be evaluated. The utilization of points remains to be convenient and photogrammetric researchers recently focused on using high level

[3] (Siriba, 2009)
[4] (Greenfeld, 2001)
[5] (Kapnias, 2008)
[6] (ASPRS), 2013)

features (linear and area feature) in various photogrammetric operations[7].Because such features increase system redundancy and consequently show the geometric strength and robustness in terms of the ability to detect blunders and local anomalies[8].

[7] (Curitiba, 2003)
[8] (Atak, 2008)

1.2 Statement of the Problem

Digital orthophotos which are photogrammetric products are being used by different sectors because of their easy interoperability. In this connection, Siriba (2009) stated that the identification of parcel boundaries using photogrammetric method has become an alternative approach to ground based surveying and has been adopted in different countries for modernizing land information registration and administration system. In recent years, much technological advancement has been made in the field of photogrammetry and the process of making maps using orthophotos. However,In principle, photogrammetric data (e.g. Orthophoto and line map) are required to be validated for positional accuracy before the use of the data for practical cadastral applications. The quality of photogrammetric data should be checked for positional errors, attribute errors, temporal errors, logical inconsistency and incompleteness, of which the positional and attributed errors are considered the most significant (Siriba, 2009). In relation to this, Greenfeld (2001) emphasized that orthophotos should undergo orientation accuracy assessment in addition to a spatial accuracy examination. Assessing the accuracy of photogrammetric data for position and orientation is not only necessary, but also mandatory for effective use of orthophoto and cadastral map for land governance and land administration.

In this regard, only few studies have been conducted in Ethiopia to investigate the positional accuracy of orthophoto and cadastral digital line maps. Sisay et al. (2018) assessed the horizontal accuracy of orthophoto and digital line map for Bahir Dar city. No published research articles are available specifically on the accuracy assessment of orthophoto and cadastral digital line maps of Addis Ababa city. This study aims to evaluate the horizontal positional accuracy of orthophoto and digital line maps of Addis Ababa city using in-situ GPS data.

1.3 Objective of the Research

1.3.1 General objective

- ✓ To assess horizontal positional accuracy of an orthophoto of Addis Ababa

1.3.2 Specific objective

- ✓ To assess the agreement between horizontal coordinates of surface points derived from orthophoto and in-situ static GPS measurements.
- ✓ To evaluate the consistence between GPS derived coordinates acquired from different processing software packages

1.4 Significance of the Study

This research can be used as a spring board for further studies for those who are interested in the area. On the other hand, the study can help users to assess the positional accuracy of an orthophoto for cadastral that going to be held in different cities within the country in a given task. Moreover, since coordinates of the reference points are determined with high precision, it can be serves as a reference values for other users.

1.5 Scope and Area of Study

The scope of this study is limited within evaluating horizontal positional accuracy of an orthophoto with that of line and checking with line features (road curves) within the study area. Determining and evaluating the accuracy of the measurement need quite stable weather condition and carefulness. Also the study covers part of Bole and Yeka sub city in Addis Ababa city.

1.6 Limitation of the Study

This research paper tries to evaluate positional accuracy of an orthophoto which needs too much time for data collection of check points that are minimum of twenty points as standard for areas less than 500km^2 because of this instruments like GPS used for data collection are difficult for easy accessibility and also the researcher tries to take longer observations time but it's difficult to take this amount of check points in shorter period of time because of this only 2hr observation time is conducted.

1.7 Thesis Structure

This paper is organized in to six chapters including reference and annex. The first chapter is an introduction which includes background of the study, statement of the problem, research objective (general and specific), and significance of the study, Scope and Area of Study and Limitation of the Study. The second chapter is literature review which is mostly discussed on describing Orthophoto which the base for line map production and also discussed different positional accuracy assessment standards used in different countries and organizations. The third chapter is description of the study area which it describes the location topography, climate, Socio Economic Characteristic of the study area. The fourth chapter is research method and materials which is discussed about data and material used, sampling design and data acquisition techniques, and also data analysis and validation techniques. The fifth chapter is results and discussions which presents output of the paper by comparing GNSS processing results of different processing software packages and assessing horizontal positional accuracy. The last chapter which is conclusion and recommendation part discussed the overall result and gives some suggestions for future work.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Introduction

Orthophotos have been used extensively in many applications today. In an attempt to map the earth's surface in a shortest possible time and provide information for monitoring and planning, Orthophotos have proved reliable as far as these are concerned. Orthophotos are one of the most important base layers of data for GIS systems. The use of Orthophotos in a GIS often implies seamless image data. The major advantage of Orthophotos is their ability to be produced in a short time to provide up-to-date information for urgent planning. They are also produced at a less expensive production price than line or vector maps.

The need therefore for quick and reliable data for many rising applications has led to the development in this technology and hence finding ways to make it better in terms of its appearance (radiometric enhancements) and geometric accuracy. Orthophotos are images that have all distortions due to camera obliquity and terrain relief removed direct measurements on them can be made just like on line maps because of this Orthophotos in combination with line maps and as a background layer in GIS techniques have made Orthophotos to be much favored recently. So when we are dealing about Orthophoto and line maps which are produced from it and it is better to recognize the accuracy of an image and maps in the real world in x , y, and z direction. Positional accuracy is "the degree of compliance with which the coordinates of points determined from a map agree with the coordinates determined by survey or other independent means accepted as accurate." All locations on maps and georeferenced images are expressed by a set of values: x- and y-coordinates for horizontal location[9]. Positional accuracy uses sampling to estimate the discrepancy between a map or image feature's coordinates or elevations and their "true" location on the earth's surface either in horizontal or vertical.

Incorporating linear features in photogrammetric applications has to address key issues. one should determine the most convenient alternative for representing straight-line features in the image. In this paper horizontal positional accuracy of line map and orthophoto are consider to be assessed and orientation of features also evaluated.

[9] Glossary of the Mapping Sciences (ASPRS and ASCE, 1994)

2.2 Photogrammetric Data Accuracy Assessment

Positional accuracy assessment is an important method of evaluating the quality of spatial dataset (Girres and Touga, 2010). It determines how closely the positions of discrete objects or features are compared to their actual locations on ground (Congalton and Green, 2008). Effective resource mapping requires accurate maps or at least maps of known accuracy. Although the positional accuracies of individual datasets are assessed, the total uncertainties of the integrated data are rarely determined.

The common types of errors in digital spatial data sets include positional error, attribute error, temporal error, logical inconsistency and incompleteness, of which the positional and attribute error are considered the most important (Shi and Liu, 2000). In this paper, positional errors are considered. Positional errors are brought about by field measurements, digitization and other processing and can be either systematic or random. The systematic errors can be eliminated by correctional procedures, while the random errors can be modeled by either using analytical or simulations approaches. The estimation of the accuracy of a geospatial dataset on the basis of the inherent systematic errors is considered.

The accuracy of a map prepared by photogrammetric techniques depends on the accuracy of the ground control and the ability to identify them on the imagery (interpretation), the scale of aerial photography and the scale of mapping. In digital photogrammetry context, mathematical models exist for estimating the expected standard error in position and height of the points (Konecny, 2008).

2.3 Linear Features for Photogrammetric Activities

With the beginning and nonstop evolution of digital photogrammetry as a result of the accessibility of high-resolution scanners and advances in digital image processing, automated procedures are becoming accepted. However, most of these procedures are still point-based. Image processing techniques allow for the extraction of numerous points with lower quality than those selected manually. Moreover, automatic matching of the extracted points in overlapping images is complicated and often unreliable procedure due to variations in the imaging system’s point of view and relief displacements in the image space. Due to the above limitations, recent photogrammetric research has been focusing on the use of linear features. This line of research has been motivated by the following facts:

Image space linear features are easier to extract when compared to distinct points. This is attributed to the nature of linear features since they represent discontinuities in the grey value function in one direction. On the other hand, point features represent discontinuity in all directions. Linear features in the image space can be extracted with sub-pixel accuracy across the direction of the edge. Images of man-made environments are rich with linear features.

Linear features allow for the incorporation of areal features through the use of their boundaries. Moreover, linear features are easier to use in change detection applications than areal features. The superiority of linear features stems from the possibility of dividing them into smaller subsets. On the other hand, breaking real features into smaller subsets is not a minor task. Mobile Mapping Systems (MMS) can economically provide accurate and current object space linear features in real time.

Linear features possess higher semantic information, which are desirable for subsequent processes (such as DEM generation, map compilation, and object recognition). On the other hand, it is hard to derive useful semantic information regarding the real world from distinct points. Moreover, geometric constraints are more likely to exist among linear features than points. This will facilitate subsequent automatic matching and object recognition activities.

Linear features increase the redundancy and improve the robustness and the geometric strength of various photogrammetric adjustment activities.

2.4 Data Accuracy Standards

This section reviews map and data accuracy standards in the United States (US), Australia and Ethiopia.

2.4.1 The US National Map Accuracy Standards

Although maps primarily impart information about position, positional accuracy standards were not introduced in the United States until the middle of last century. Prior to this, indeed before 1900, the positional accuracy of maps made of US cities was controlled by (a) a document specifying the method of survey, and the extent and distribution of the survey control network, and (b) checking of the work as it progressed (Marsden, 1960). In a few cases mathematical tolerances for the positioning of well-defined features were specified. In the early 1900s similar quality control methods were used by the United States Geological Survey (USGS) for the production of small scale quadrangle maps, albeit without the comprehensive check as mapping progressed. Marsden (1960, pp. 428-429) notes: “the accuracy of maps produced during this period depended largely on the skill of the individual topographers [and] it became common practice to evaluate the accuracy of any given topographic map according to the professional reputation of the topographer who made it.”

The development of the US National Map Accuracy Standards (NMAS) was prompted by the revolution in mapping brought about by photogrammetry. In the 1930s the use of photogrammetry for mapping escalated from its beginnings in about 1920 but “did not receive wholehearted acceptance within the surveying and mapping profession for the next several years” (Marsden, 1960, p.428). Mapping professionals did not accept that photogrammetry “could duplicate a skill acquired through many years of field experience” (Marsden, 1960, p.428). At the same time the development of standards received impetus from other quarters. With private firms undertaking contract mapping work there was a need for a standard by which to judge their work. In addition, special purpose maps by government and private entities were being produced and there needed to be a standard by which to judge whether they could be included in the standard quadrangle maps produced by the USGS (Marsden, 1960).

In 1937 the American Society of Photogrammetry formed a committee to draft accuracy specifications. Marsden (1960, p.429) notes: “Although most of the members of the profession agreed that accuracy standards were urgently needed, they held widely diverse opinions as to what those standards should be based on. Some suggested that the accuracy of a map should be judged by the amount and distribution of the control. Some thought that the degree of accuracy should vary with the scale – that the larger the scale of the map, the greater the relative accuracy should be. Contrariwise, others thought that the smaller the map scale, the greater the relative accuracy should be. Others suggested that there should be a single set of accuracy standards for all maps, regardless of scale”

Two years later in 1939, the committee published its map accuracy specifications in the journal Photogrammetric Engineering, and invited criticism. The committee believed that by publishing the specifications, other organisations would then take the lead in developing national standards, and indeed, in 1940 a committee under the Federal Board of Surveys and Maps was formed to develop an accuracy standard. This committee ultimately developed a standard based on the method of production. They specified a different standard for maps produced from photogrammetry to those produced from field survey. In 1940 this specification was tentatively adopted, but the standard lapsed and was never fully ratified (Marsden, 1960).

In 1940 an appointee of the United States Director of the Bureau of the Budget: "arranged a series of conferences with representatives of the mapping profession and the United States Federal mapping agencies with the objective of developing standards applicable to maps at all scales and which would be clear, concise, and free of ambiguity." (Marsden, 1960, p.431)

They issued the original version of the "United States National Map Accuracy Standards" in 1941, which was similar to that issued by the American Society of Photogrammetry in 1939. The standard was reviewed in 1943 and 1947. In its final form there was only one class of accuracy standard, and it was made clear that each agency was responsible for the conformance testing of its maps, and that there was no compulsion to conform. The United States National Map Accuracy Standards state: "For maps on publication scales larger than 1:20,000, not more than 10 percent of the points tested shall be in error by more than 1/30 inch, measured on the publication scale; for maps on publication scales of 1:20,000 or smaller, 1/50 inch. These limits of accuracy shall apply in all cases to positions of well-defined points only. Well-defined points are those that are easily visible or recoverable on the ground, such as the following: monuments or markers, such as bench marks, property boundary monuments; intersections of roads, railroads, etc.; corners of large buildings or structures (or center points of small buildings); etc." (U.S. Bureau of the Budget, 1947, p.1)

The standards further state that any map may be tested for compliance with the standard, with those found compliant having a note on the legend "as follows: 'This map complies with National Map Accuracy Standards.'" (U.S. Bureau of the Budget, 1947, p.1).

2.4.2 Other US map accuracy standards

Other US standards for positional accuracy similarly defined accuracy in terms of well-defined points. For instance, the American Society for Photogrammetry and Remote Sensing (ASPRS) Accuracy Standards for Large-Scale Maps, 1990, defined accuracy tolerances for engineering and special purpose maps at 1:20K scale or larger in terms of well-defined points (Chrisman, 1991).

2.4.3 The US National Standard for Spatial Data Accuracy

The US National Standard for Spatial Data Accuracy (NSSDA), released by the US Federal Geographic Data Committee (FGDC) in 1998, updated the NMAS with input from the ASPRS accuracy standards. The intent was for the new standard to encompass digital data as well as paper maps, as "it was realised that map-dependent measures of accuracy, such as publication scale and contour interval, were not fully applicable when digital geospatial data can be readily manipulated and output to any scale or data format" (Federal Geographic Data Committee, 1998, p.3-2).

However they too failed to address the positional accuracy of features other than well-defined points, stating that "Horizontal accuracy shall be tested by comparing the planimetric coordinates of well-defined points in the dataset with coordinates of the same points from an independent source of higher accuracy" (Federal Geographic Data Committee, 1998, p.3-4).

2.4.4 The Australian "Standards of Map Accuracy"

The Australian "Standards of Map Accuracy", released by the National Mapping Council of Australia, in 1953, six years after the release of the third edition of the US NMAS, was almost a replica of NMAS. It too refers only to well-defined points.

"(a) For maps published at scales larger than 1:20,000. Not more than 10 per cent of points tested shall be in error by more than 1/30th of an inch measured on the publication scale.

(b) For maps published at scales of 1:20,000 or smaller. Not more than 10 per cent of points tested shall be in error by more than 1/50th of an inch measured on the publication scale. These limits of accuracy shall apply in all cases to positions of well-defined points only. 'Well defined' points are those that are easily visible or recoverable on the ground." (National Mapping Council of Australia, 1953, p.1)

There has been no update to this Australian standard to accommodate digital spatial data. Although guidelines for metadata have been developed, such as the Spatial Information Council of Australia and New Zealand (ANZLIC) Metadata Guidelines, these guidelines define a positional accuracy reporting protocol, rather than a desired level of accuracy, and a method of measuring accuracy.

While other standards for positional accuracy exist in Australia, such as those specified in state and territory survey coordination regulations for survey plans, they target special purpose plans and not the general purpose maps that fell under the, now disbanded, National Mapping Council, and which are of concern here.

2.4.5 Ethiopian "Standards of Map Accuracy"

In the Ethiopian context, the accuracy requirement of the horizontal position according to the National Mapping Agency (EMA) is ±30 cm at the scale of 1:2,000, which is the recommended scale for urban areas (Ministry of Urban Development, Housing and Construction, 2015, page 10). This corresponds to two pixels at a Ground Sample Density (GSD) of 15 cm. The accuracy of the vertical position is ±45 cm, likewise corresponding to three pixel.

CHAPTER THREE

3. DESCRIPTION OF STUDY AREA

3.1. Location

Addis Ababa city is located in central part of Ethiopia. Geographically, it is located at the geographic coordinates of 9^{o} 1' 48’’North latitudes and 38^{0} 44' 24’’ east longitudes. The project area is part of Bole and Yeka sub city eastern part of Addis Ababa city. The study area is part of the grand expansions of the city in recent years and serves as a political center as well as hosts many international institutions including African Union. For administrative purpose, the city is classified in to ten sub cities and around 114 woredas.

The city is now serving as a center of various social, economic and political activities to the country and international organizations. There are many construction works undertaken in the city. As a result of the expansion and development of the city as well as the availability of job opportunities, it attracts many rural and urban people.

Addis Ababa is one of and the first areas in Ethiopia that has manifested very fast changes in land use (from agricultural fields to built-up environment, both formally and informally). The transformation of local peri-urban land rights into urban leasehold rights through the expropriation decisions of the government is more rapid and extensive in the peri-urban areas of Addis Ababa as well as the inner city areas. This issues is becoming critical in Addis Ababa city than any other cities within the country, due to the increasing demand for land for urban space.

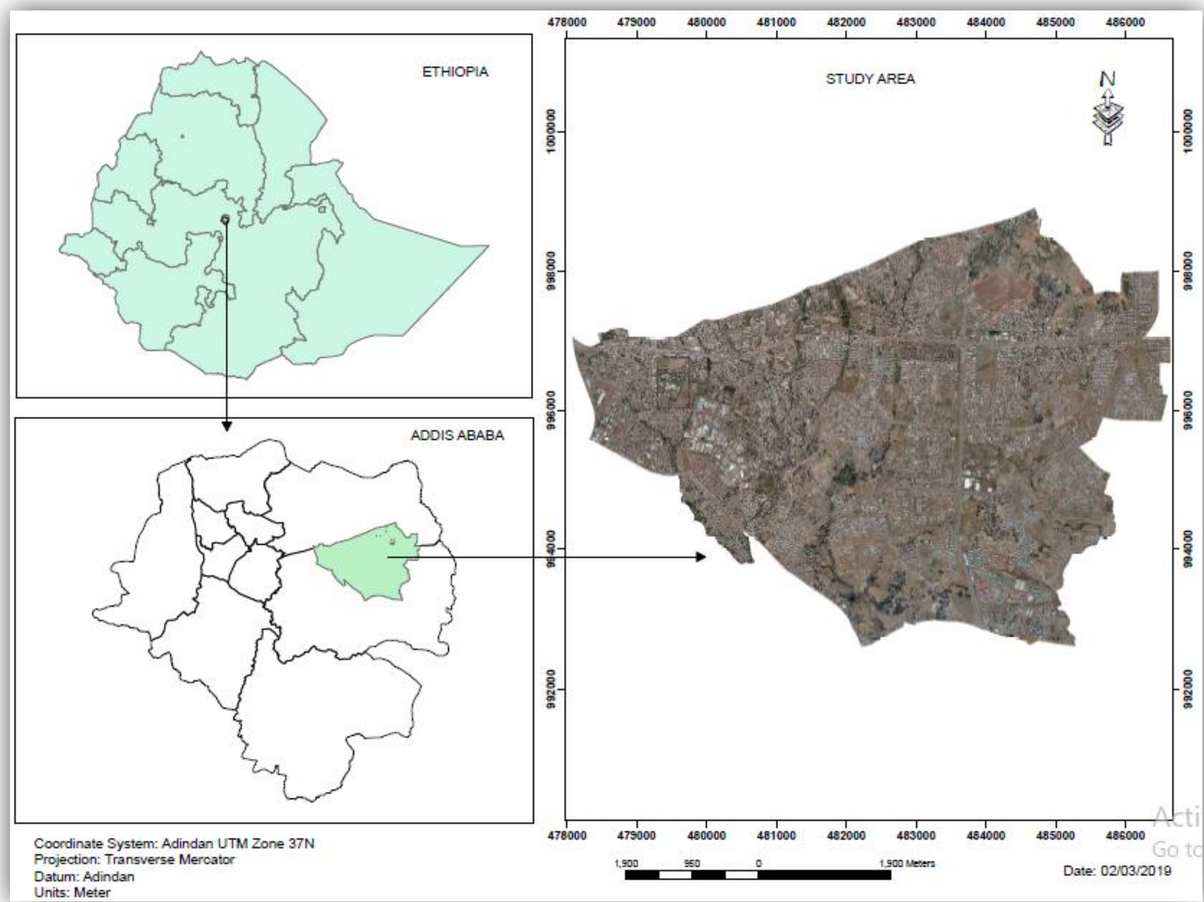

Fig 3.1 Project area and ground control points with in the study area

3.2. Topography

Addis Ababa lies at an elevation of 2200 meters and is a grassland biome, located at 9^{o} 1' 48'' N, 38^{0} 44' 24'' E. The city lies at the foot of Mount Entoto and forms part of the water shade for the Awash Basin. From its lowest point, around Bole International Airport, at 2326 meters above sea level in the southern periphery, Addis Ababa rises to over 3000 meters around the Entoto Mountain to the north. The city experiences a significant topographic undulation (Fig 3.2 and Fig 3.3).

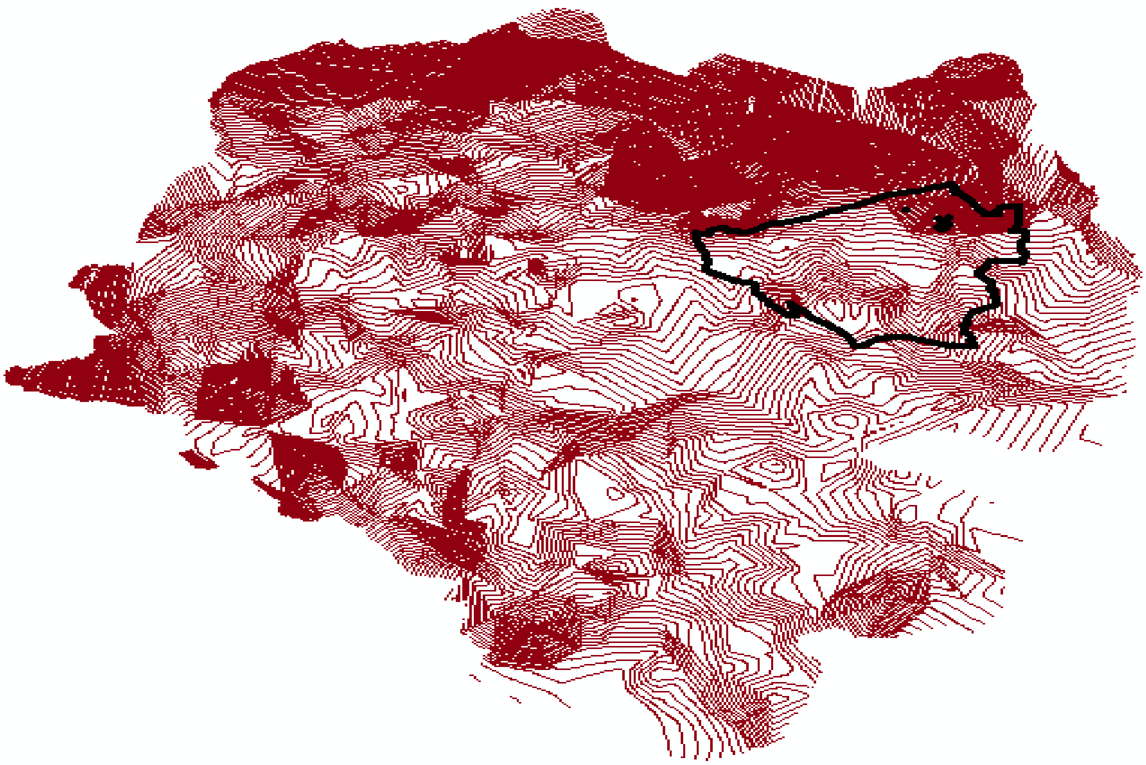

Figure 3.2- Contour map of Addis Ababa with contour interval 5 meters (Extracted from gcp's)

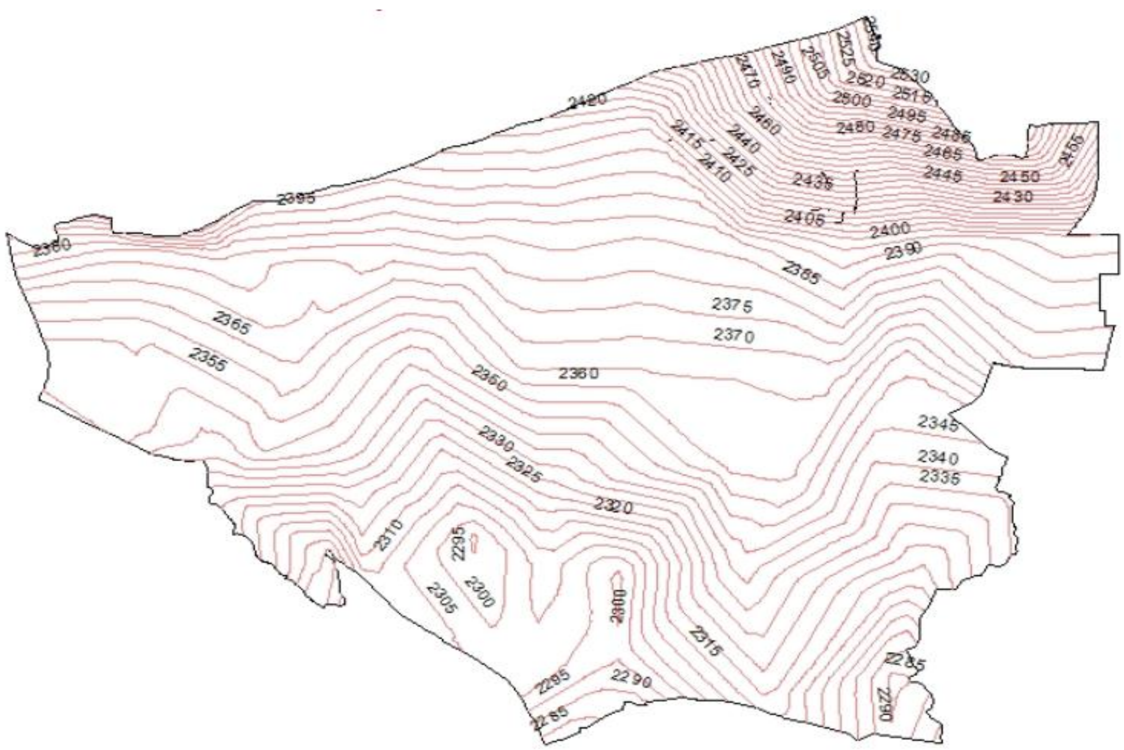

Figure 3.3 - Contour map of study area with contour interval 5 meters

3.3. Climate

Addis Ababa has a sub-tropical highland climate. The city has a complex mix of highland climate zones, with temperature differences of up to 10 °C (18 °F), depending on elevation and prevailing wind patterns. The high elevation, moderates temperatures year-round, and the city's position near the equator means that temperatures are very constant from month to month. As such the climate would be maritime if its elevation was not taken into account, as no month is above 22 °C (72 °F) in mean temperatures.

Mid-November to January is a season for occasional rain. The highland climate regions are characterized by dry winters, and this is the dry season in Addis Ababa. During this season the daily maximum temperatures are usually not more than 23 °C (73 °F), and the night-time minimum temperatures can drop to freezing. The short rainy season is from February to May. During this period, the difference between the daytime maximum temperatures and the night-time minimum temperatures is not as great as during other times of the year, with minimum temperatures in the range of 10–15 °C (50–59 °F). At this time of the year, the city experiences warm temperatures and a pleasant rainfall. The long wet season is from June to mid-September; it is the major winter season of the country. This period coincides with summer, but the temperatures are much lower than at other times of year because of the frequent rain and hail and the abundance of cloud cover and fewer hours of sunshine. This time of the year is characterized by dark, chilly and wet days and nights. The autumn which follows is a transitional period between the wet and dry seasons.

The highest temperature on record was 32 °C (90 °F) 27 August 1996, while the lowest temperature on record was 0 °C (32 °F) on 23 November 1999.

Table 3.1- Climate data for Addis Ababa

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation total in mm	13	30	58	82	84	138	280	290	149	27	7	7
Average max. temp in 0c	24	25	25	25	25	23	21	21	22	24	23	23
Average min temp in 0c	7	9	11	11	11	11	11	11	11	9	7	7

Source: National Metrological agency of Ethiopia

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Average high ^{O}C (^{O}F)	23.5 (74.3)	24.5 (76.1)	25.4 (77.7)	24.8 (76.6)	25.2 (77.4)	23.4 (74.1)	20.7 (69.3)	20.7 (69.3)	21.7 (71.1)	22.7 (72.9)	23.0 (73.4)	22.9 (73.2)	23.2 (73.8)
Daily mean ^{O}C (^{O}F)	15.4 (59.7)	16.6 (61.9)	17.9 (64.2)	17.9 (64.2)	18 (64.4)	17 (62.6)	15.9 (60.6)	15.8 (60.4)	16.2 (61.2)	15.7 (60.3)	14.8 (58.6)	14.9 (58.8)	16.3 (61.4)
Average low ^{O}C (^{O}F)	7.4 (45.3)	8.7 (47.7)	10.5 (50.9)	11.1 (52.0)	10.8 (51.4)	10.6 (51.1)	11.1 (52.0)	11 .0 (51.8)	10.7 (51.3)	8.7 (47.7)	6.7 (44.1)	7.0 (44.6)	9.5 (49.2)
Average rainfall mm (inches)	13 (0.5)	30 (1.2)	58 (2.3)	82 (3.2)	84 (3.3)	138 (5.4)	280 (11.0)	290 (11.4)	149 (5.9)	27 (1.1)	7 (0.3)	7 (0.3)	116.5 (45.9)

Source: World meteorological organization

3.5. Socio Economic Characteristic of Addis Ababa

3.5.1. Population and Demographic Characteristic of Addis Ababa

As of the latest 2007 population census conducted by the Ethiopian national statistical authorities, Addis Ababa has a total population of 2,739,551 urban and rural inhabitants. For the capital city, 662,728 households were counted living in 628,984 housing units, which results in an average of 5.3 persons to a household. In the previous census, conducted in 1994, the city's population was reported to be 2,112,737, of whom 1,023,452 were men and 1,089,285 were women. At that time not all of the population was urban inhabitants; only 2,084,588 or 98.7% were. For the entire administrative council there were 404,783 households in 376,568 housing units with an average of 5.2 persons per household.

In view of its improvement in urbanization and economic infrastructural development, the government has conducted airborne photogrammetric survey to capture land parcels and other linear cadastral information seamlessly with the high resolution. The town consists of 40,000 developed lands and including natural and man-made lands including green areas, road (straight and curve), fountains, built up areas (residential, commercial, and mixed use), Squares, Open and vacant parcels, Stadium, wetlands, forests, Churches, Mosques etc. Now, this gives essentially a new opportunity to validate the horizontal, vertical position, and orientation linear features accuracy of Orthophoto and its digital line map.

CHAPTER FOUR

4. RESEARCH METHOD AND MATERIALS

4.1. Data and Material Used

This study used orthophoto acquired from aerial photogrammetric survey carried out over the area of Addis Ababa. The photogrammetric surveying was conducted by German Air Survey with standard aerial camera at 1:2,000 scale and 20cm Ground Sample Distance (GSD). Orthophoto are obtained from the Addids Ababa Land Holding project office . Originally, the whole orthorectification process and the extraction of digital line map was carried out by Hansa Luftbild Besides, this study used in-situ ground based GNSS data taken at selected spatial locations from orthophoto. Addis Ababa city orthophotos and line map, which used as a base for cadastral work within the city taken by Hansa, with an accuracy of 20 cm on ground sample distance. The data are get from Addis Ababa city Administration landholding adjudication project office. And instruments used for this work includes SOKKIA GRX1 GPS instruments were used to take static measurement for establishment of control Points also for GPS data processing Leica Geooffice was used and ArcGIS software also used for data manipulation.

4.2. Overview of Orthophoto Used in this Study

Digital orthophoto of Addis Ababa city was generated using three stage processes. The first stage controls orientation change to define the position and angular orientation of the camera that captured an image. The second stage uses aerial triangulation creates Digital Terrain Model (DTM) while the third stage used DTM for orthophoto rectification and mosaicking. Orthophoto rectification is the process of reducing geometric errors inherent within photography and imagery due to camera and sensor orientation, systematic error associated with camera and sensor, topographic relief displacement and earth’s curvature. The digital orthophoto was generated from 295 scenes covering the whole area of Addis Ababa (Fig. 4.1). There are around 670 primary GCP’S in the city, but some of them were used for photogrammetric exterior orientation changes.

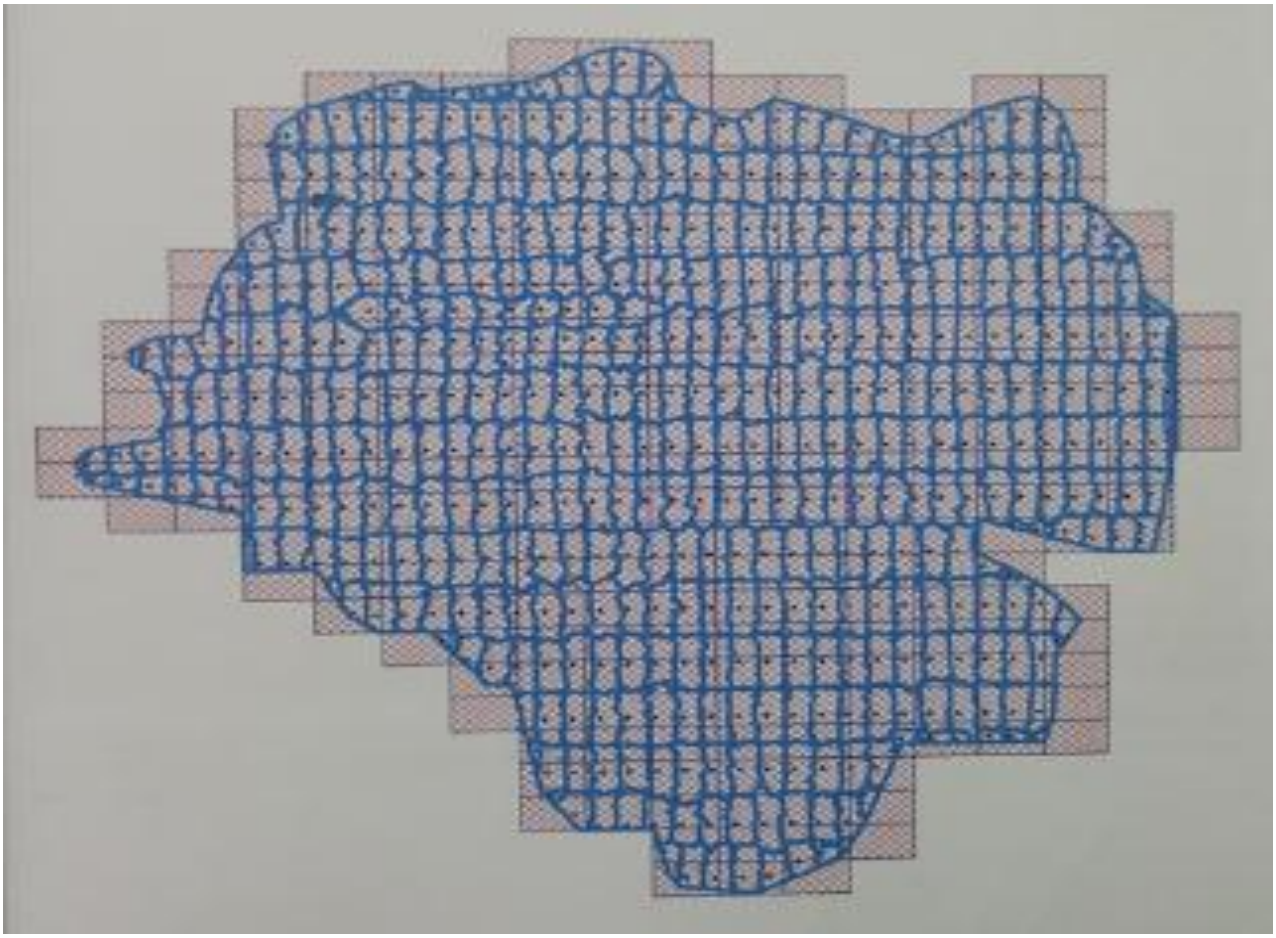

Figure 4.1 - Scenes captured the whole Addis Ababa

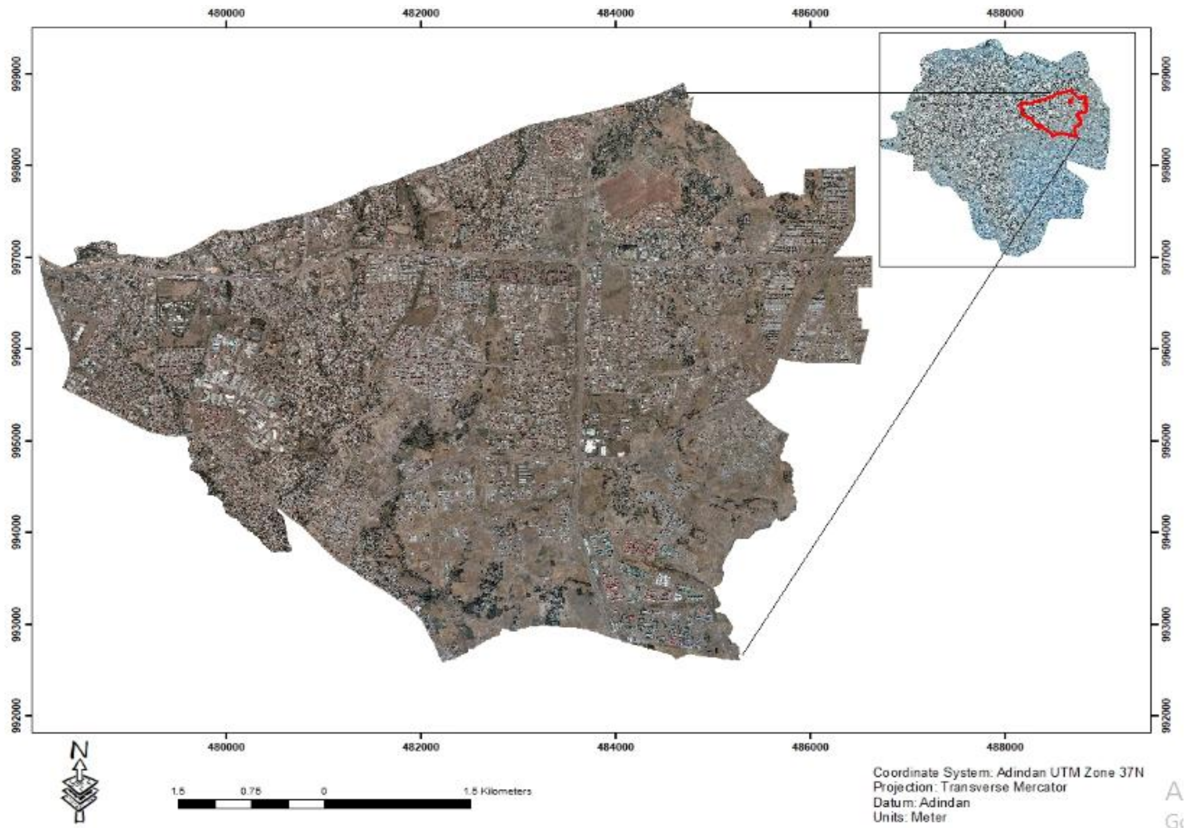

Figure 4.2 - Orthophoto of study area / Addis Ababa city orthophoto

Digital line map is extracted from orthophoto by digitizing each feature as observed from it. The output features are vector data that represent real world features. The digital orthophoto and line map used in this study are projected into Universal Traverse Mercator plane coordinate system with unit in meter, using the 1880 Clarke ellipsoid as a datum. Orthophoto and digital line map are important for the development of cadastral system and other applications. The vector datasets in line map include administrative boundary, GCP's, land parcel, line features, polygon features. Feature extraction from orthophoto involves the task of tracing points on/along interested features (Fig.4.2). For instance, road center line can be extracted by tracing the center line of the road features from orthophoto. The main problem associated with the extraction of features is to get height information; only the x- and y-components are available. There is building height, but it is the averaged to all parts of the building.

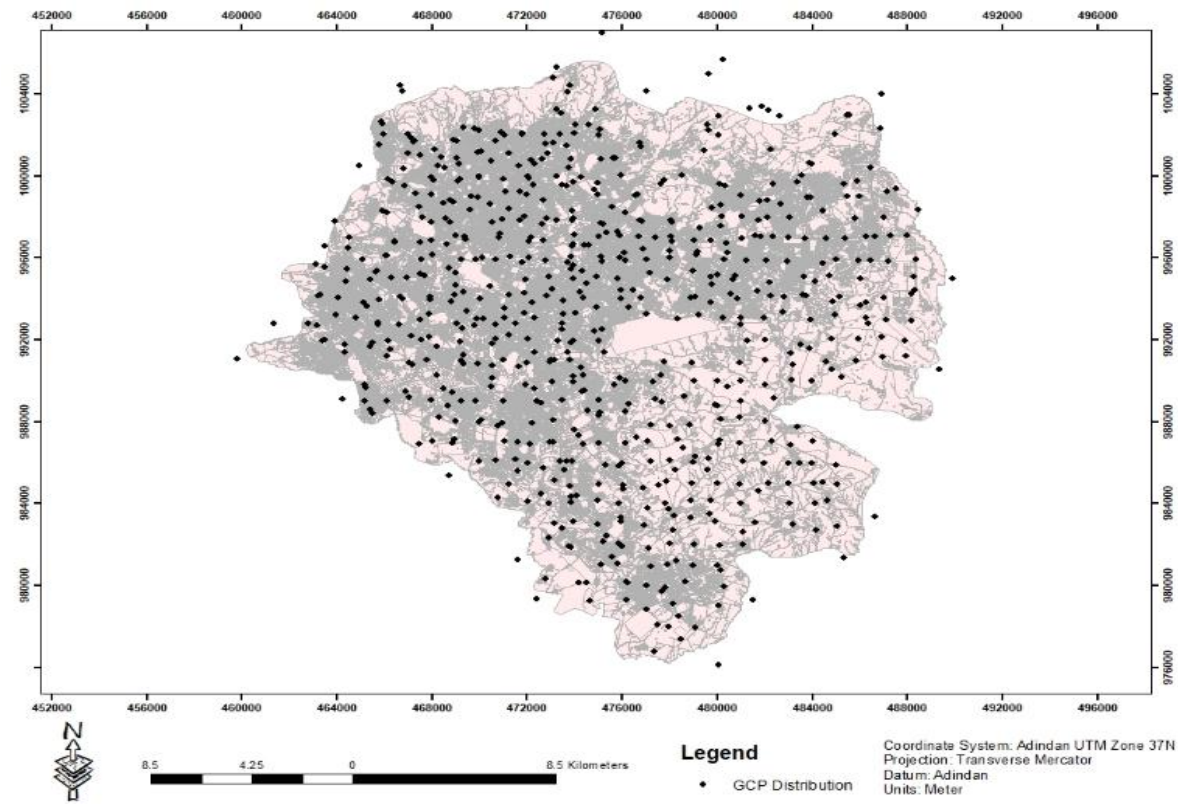

Figure 4.3- Line map of Addis Ababa city with 670 GCP's

4.3. Check Point Sampling Design and Sample Size

In this paper the sampling design component of accuracy assessment is singled out for detailed attention. Estimating errors in the positional coordinates of spatial objects requires the comparison of coordinates (x,y) of identical sample locations from photogrammetric data with the corresponding real spatial objects on the surface of the Earth. Planning the sampling design depends on three fundamental features.1) the accuracy objectives; 2) the desirable sampling design criteria; 3) the strength and and weakness of basic and complex sampling designs relative to these objectives and criteria. Considering this Cluster sampling technique is used due to the fact that the selection and distribution of the Checkpoints (CP's) primarily depend on the visibility and recoverability of well-defined points on orthophoto by ground observation using GPS receiver. Besides, the samples shall be representative in view of topographic and land use/land cover variation. The appropriate locations of CP's could be canal corners, roads junctions and centers of utilities, etc.

The selection of check points and its spatial distribution followed the 1998 international standard of the National Standard for Spatial Data Accuracy (NSSDA)/FGDC. Besides, the sizes of the samples were determined on the basis of ASPRS standards. The 1998 NSSDA of the United States of American standard provides excellent guidance on positional accuracy sample design, estimation of sample size and sample selection criteria. The standard was developed by Federal Geographic Data Committee consists of different departments. The followings are the basic requirements set for sampling design and selection as per the standard of the 1998 NSSDA. The reference data independence from the data being tested is critically important. The first criteria is about the accuracy of the reference data - the reference data should be at least three times accurate than the data being assessed. The second criteria state that a minimum of 20 sample points are required to test positional accuracy with reliable statistical rigor as the size of the study area is less than $500km^2$. The third criteria states that the selection of the samples must consist of “well-defined points” that are easily visible or recoverable on the ground. Whereas, the fourth criteria states that sample points should be well distributed across the experimental site, and represent the full variety of topography

The 1998 ASPRS’s standard also explained that the selection of the number of checkpoints depend on the area of the experimental site. For instance, the study site having an area ≤ 500, 501-750, 751-1000 in square kilometers require to have the corresponding number of checkpoints 20, 25, and 30 respectively. The 1998 NSSDA standard recommended that systematic sampling methods that ensure sample points are well distributed. Minimum of 20% of the sample points are allocated to each quadrant, no two points should be closer than d/10, where d is the diagonal dimension of the map or image. For this study, the sample locations are determined on the basis of the standards and criteria as discussed in the above section. The spatial objects are identified and selected by visualizing the digital orthophoto by zooming in and out in ArcGIS platforms.

In connection to this, the study combines the 1998 NSSDA and ASPRS with Greenwalt and Schultz’s (1968) equations to practice horizontal positional accuracy assessment of digital orthophoto and its extracted features. In the Ethiopian context, the accuracy of the horizontal position according to the National Mapping Agency (EMA) is ± 30cm (at the scale of 1:2,000), while the accuracy of the vertical position is 0.25 or ¼ of the contour interval (i.e, ± 50 cm for

1:500 scale, $\pm$ 100cm for a 1:1000 scale, $\pm$200cm for a 1:5000 scale $\pm$ 200cm for a 1:5000 scale.

4.4. GPS Data Acquisition Technique for Checkpoint Measurement

4.4.1. Reconnaissance Survey

An important step in the data acquisition is field reconnaissance. Reconnaissance consists of checking the field project sites for their suitability of GPS survey, availability of control points and logistic requirements. Through field reconnaissance obstruction and interference may be identified and avoided by alternative site selection or through establishment of eccentric stations. To avoid satellite signal blockage, ideally a site should be obstruction free in all directions above 15^0 elevations. In less than ideal conditions where some obstructions do exist, successful positioning is possible if a sufficient number of satellites with adequate geometry can be tracked. Interference of GPS signals is mainly due to multipath effect. Avoiding the site near artificial structures and metallic objects or any other reflected objects reduce it. During reconnaissance, control stations planned for use should be checked to ensure that they are in stable conditions and are suitable for GPS observations. so during reconnaissance the main concerns that are checked: i) the existence of identified spatial objects on the earth's surface, ii) the suitability of the existing checkpoints for GPS observations, that is, to assess satellite visibility; the satellite obstruction for the twenty checkpoint location were nearby building and tree features., iii) the representativeness of samples geometry distribution, iv) the consideration of topographic variation,. The distribution of check points in each quadrant is designed on the basis of creating well defined spatial geometry sample points. If control is unavailable, one may desire to establish a new point through a conventional static GPS survey using control in the surrounding areas. The topographic nature of the orthophoto used in this study is a mix of flat ground and slightly undulating elevation so some areas had fewer checkpoints because it was more difficult to identify "well-defined points" from the orthophoto. The checkpoints were located mostly near easily accessible sites representing elevation differences. The geometrical distributions of the checkpoints that are identified for this study (Fig. 4.4) are not equally spaced, but they are well representative. Besides, the sample size is very dependent on topography and area; in this case, twenty samples were collected by using Sokkia GRX1 Differential GPS receiver for assessing positional accuracy. Also during Reconnaissance much needed information about logistic requirements the mode of transportation and time required walking from one point to other point have significant implications for both the cost and logistics of a given survey. Similarly if any constraints identified can be solved accordingly in advance.

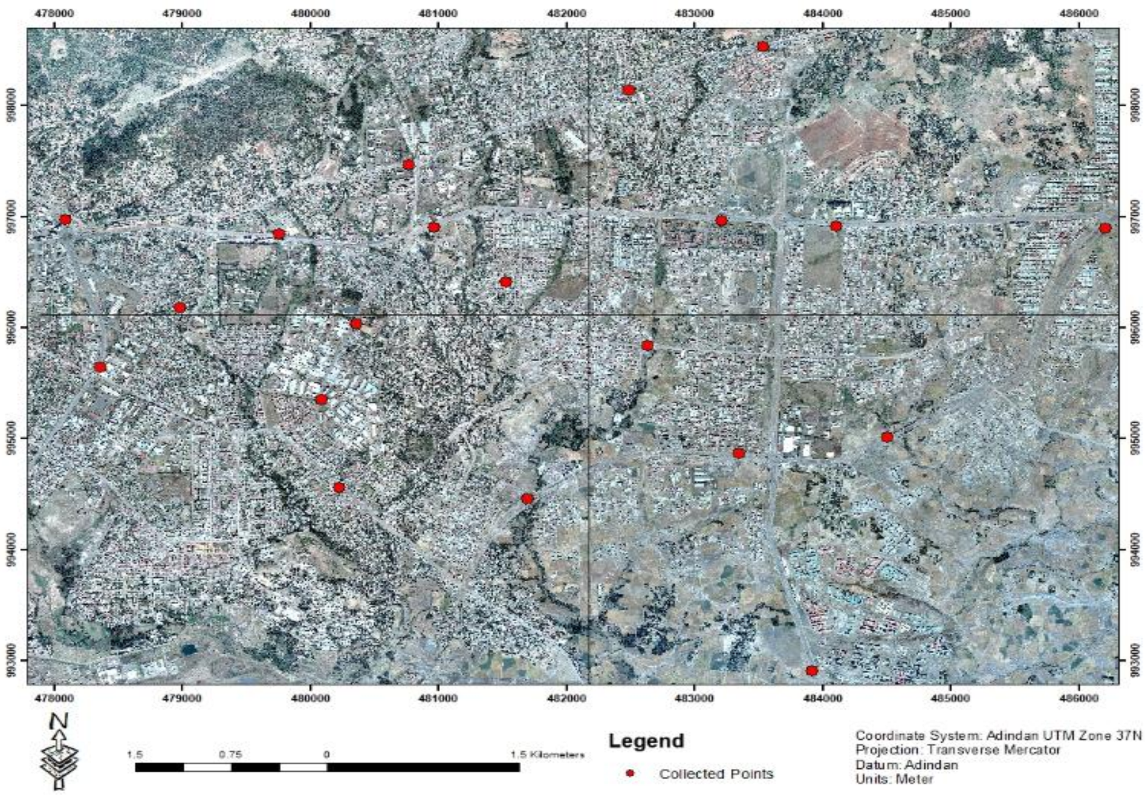

Figure 4.4- Refined checkpoints location based on detailed field reconnaissance survey

4.4.2. GPS Planning

Planning and design of surveys using GPS are required to ensure the efficient execution of operations within acceptable standards. GPS positioning techniques vary significantly and it follows different procedures; Planning and Preparation, Field Operation and Data Processing. Validation and reconnaissance form an integral part of the planning and preparation phase. Also selecting check points that have good satellite visibility by considering different parameteres like Dilution of Precision (DOP) can be considered in this step. GDOP helps to judge the geometry of the satellite constellation during measurement. A low GDOP indicates good geometry. A high GDOP value means that the satellite constellation is poor. The lower the values of GDOP, the better accuracy of GNSS derived coordinates of the check points can be achieved.

4.4.3. GPS Observation

With good planning and preparation, the field operations should be relatively smooth. GPS observation time, other interrelated and important parameters to be considered. Several factors depend the observation time required for an accurate result in post-processing: baseline length from the reference station, number of satellites, satellite geometry (GDOP) and atmospheric effect. Static observations for positional accuracy assessment was the typical surveying method used in this research was (Fig. 4.5). The required observation time is mainly a function of the baseline length and atmospheric effect. GNSS measurement was taken for the duration of 2 hours at each checkpoint at data logging rate of 30 seconds. During measurement sessions, the availability of both GLONASS and GPS satellites varies from 7 to 9 on average.

Figure 4.5 - Static GPS observation at one of the twenty CP’s location

GNSS/GPS observation is used to assess the spatial accuracy of the digital Orthophoto. The uncertainty of GNSS coordinates of the checkpoints should be at least three times better than the accuracy of the same coordinates derived from Orthophoto. The ground based in-situ measurement used SOKKIA GRX1 for long observation period to increase the accuracy of the coordinates of the check points – measurement. Furthermore, post-processing of GNSS data used network of Continuously Operating Reference Stations (CORS) and International GNSS Service (IGS) stations to improve the positional accuracy of the checkpoints. According to Greenfeld (2001), the best accuracy of GNSS coordinates for check points can range between 6 mm to 15 mm, and photogrammetric data acquired with level of accuracy can be legitimately used for cadastral boundary demarcation. In relation to this, the study used 20 checkpoints to assess the spatial accuracy of orthophoto.

The GNSS data acquired at twenty check points was processed with a tie to local and regional GPS reference stations. For example, ADDIS CORS station located at Natural science campus in Arat killo,Addis Ababa University (Fig.4.6). The respective baselines among checkpoints and between checkpoints and GPS reference stations were considered. The ground based in-situ measurement used SOKKIA GRX1 for long observation period to increase the accuracy of the coordinates of the checkpoints.

4.5. GPS Data Post Processing and Adjustment

Post processing is used in Differential GPS to obtain precise positions of unknown points by relating them to known points such as survey control points. It uses network of Continuously Operating Reference Stations (CORS) and International GNSS Service (IGS) stations to understand the positional accuracy of the checkpoints. GPS data processing software computes baselines using simultaneous measurement data from two or more GPS receivers. The baselines represent a three-dimensional line drawn between the two points occupied by each pair of GPS Receives. The post processed measurements allow more precise positioning, because most GPS errors affect each receiver nearly equally, and therefore can be cancelled out in the calculations. (Khaled, 2015). The uncertainty of GNSS coordinates of the checkpoints should be at least three times better than the accuracy of the same coordinates derived from orthophoto or/and digital line map.

4.5.1. GPS Data Processing by Spectrum Survey office

The technique of differential processing of GPS data using Spectrum Survey office software require to establish differential baseline network by setting the field GPS measurement stations as rover mode and RINEX files from the network as the reference mode. For this purpose, the Receiver Independent Exchange Format (RINEX) file and Broadcast (BRDC) file have been downloaded from those stations via Internet[10] and imported into Spectrum Survey office by using licensed Dongle key to activate the RINEX data format. On the other hand, the in-situ DBX GPS data format was converted into RINEX format and imported into Spectrum Survey office processing software in system 1200/900 raw data format. Once this setup is completed the differential processing is usually conducted in a least square adjustment sense.

The general steps to compute the baseline by Spectrum Survey office GPS data processing software are; the first step is data organization, i.e., project creation. The next step is to import and set the project files. Organization of the Spectrum Survey office working desktop is also different from the other packages. It is divided into a number of tabs. The most important tabs for GPS measurements processing are: View/Edit, GPS-Proc, Adjustment, Points, Antennas, and Results.

This study used local Cors station for processing GPS data using Spectrum Survey office. To process GPS data with a tie to local GPS network as a reference station to minimize the error distribution across the twenty checkpoints. The GNSS data acquired at twenty checkpoints was processed with a tie to reference stations namely ADDIS CORS station located at Natural science campus (Fig.4.6).

[10] http://igs.bkg.bund.de/file/rinexsearch

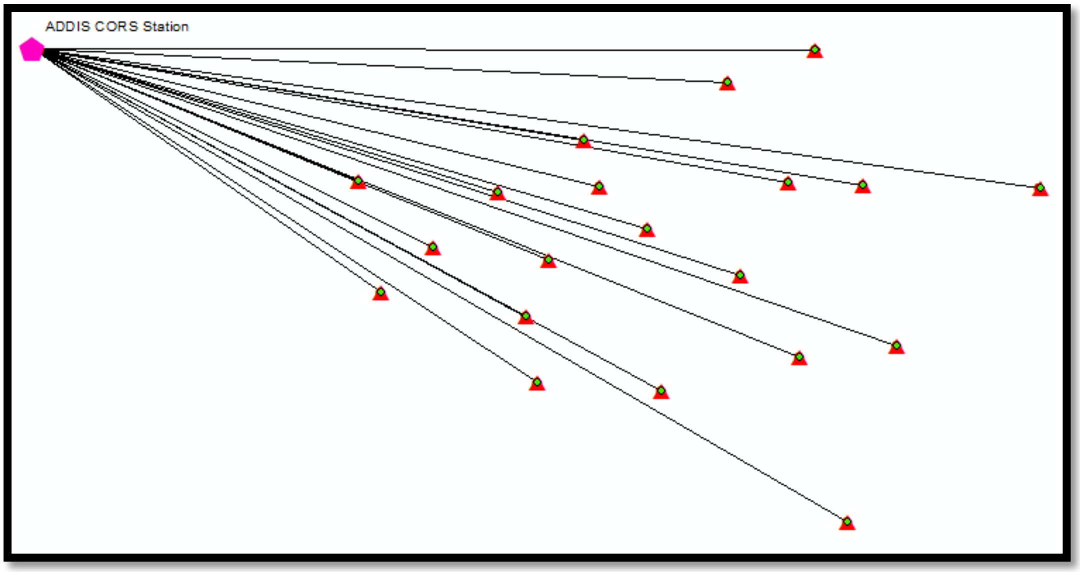

Figure 4.6- Twenty new GNSS points and reference stations

The distances between the checkpoints are computed by using baseline estimator tool in ArcGIS platform. The observation time for those reference stations is at least equal to the GPS observation time for the sake of having common observation window during post processing). The RINEX, BRDC, Ephemeris data were downloaded from IGS and CORS stations[11] for the computation of precise position coordinates.

During post processing the checkpoints are connected to each other having the same observation time, and the GPS observations were expressed in terms of longitude, latitude and ellipsoidal height defined in geodetic coordinate system.

4.6. Method of Coordinate Transformation from WGS84 to Adindan/Clarke1880

The GPS data is defined with respect to the 1984 World Geodetic Reference System (WGS84), while the coordinates of orthophoto are referenced to Adindan UTM 37N so comparison of GPS coordinates of the checkpoints with the corresponding coordinates acquired from orthophoto requires coordinate transformation. Because the GPS coordinates and the coordinates extracted from orthophoto are defined in different coordinate reference system. The transformation parameters between the two datasets require correct parameters from WGS84 to Adindan UTM

[11] http://igs.bkg.bund.de/file/rinexsearch

37N or vice versa (Tables 4.1 and 4.2). In relation to this, Fig. 4.7 illustrates the overall process of positional accuracy assessment.

Table 4.1 Transformation parameters from Adindan to WGS 84

Parameters	Dx	Dy	Dz
Values	162	12 m	-206 m

Table 4.2 Transformation parameters from WGS 84 to Adindan

Parameters	Dx	Dy	Dz
Values	-162	-12 m	206 m

Where Dx, Dy &Dz are translation parameters and Rx, Ry &Rz are rotation parameters, S scale factor

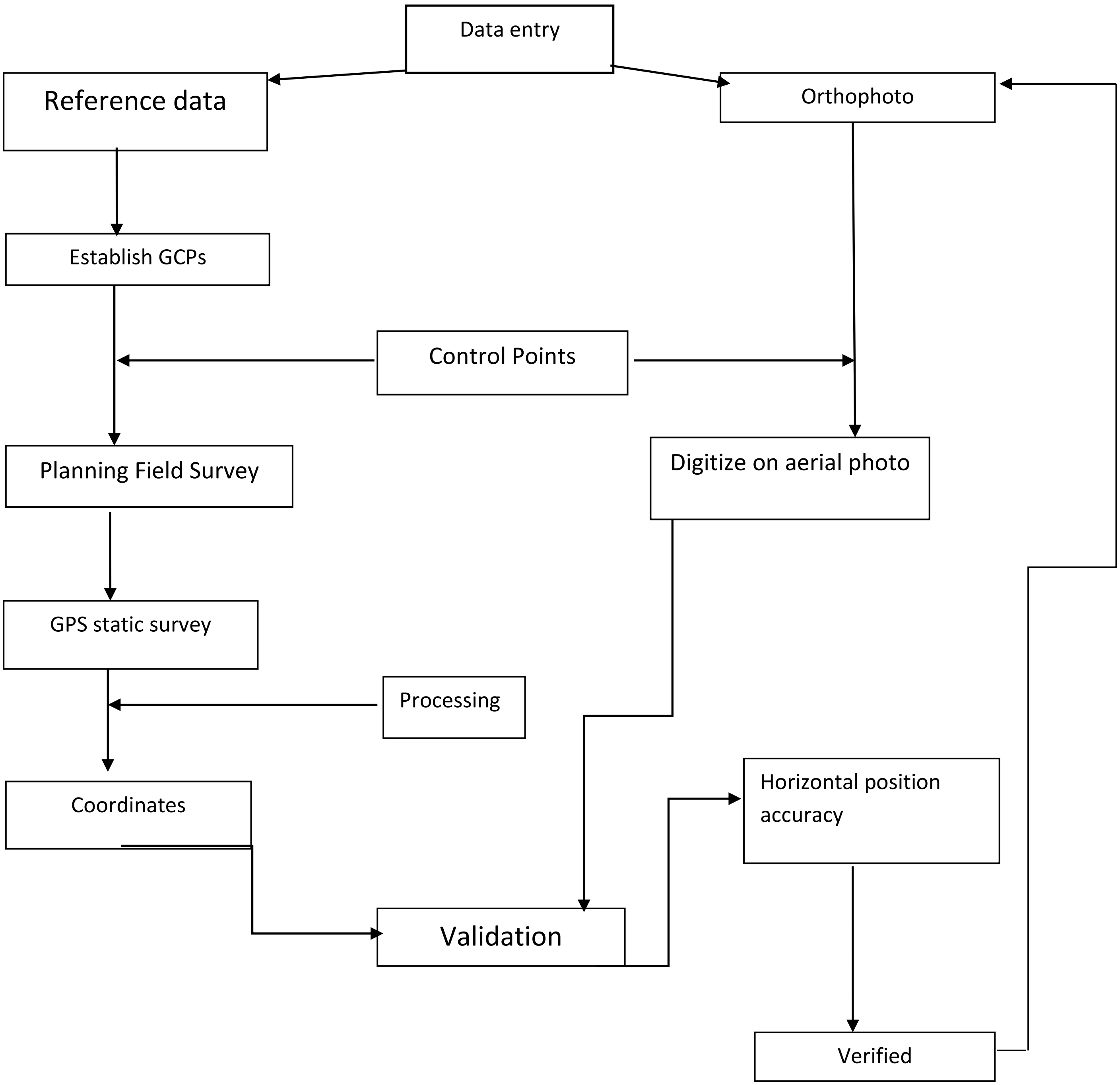

Figure 4.7- Schematic Diagram of Workflow

4.7. Data Analysis and Validation Techniques

Positional accuracy of aerial photogrammetry has been assessed by comparing the coordinates of sample points on a map against the coordinates of the same points derived from a ground survey or some other independent source deemed to be more accurate than the map (Congalton and green, 2009). In other words, all positional accuracy parameters are estimated by comparing reference coordinates and the map or image/photo coordinates or elevations of the dataset being assessed at each sample location. Different statistical approaches such as Root Mean Square Error (RMSE), Standard error, and 95 % confidence limit are used to estimate the mismatch between the accuracy of orthophoto and digital line map with the ground truth data. The quality of the CP's coordinates was evaluated by computing the RMSE and standard deviation directly from the double differencing GPS data processing technique.

According to the 1989 NSSDA's standard, the equation for the average horizontal error or horizontal RMSEh is calculated from the errors of the individual test sample points using equation 4.1.

$$\mathrm{RMSE_h} = \frac{\sqrt{\sum_{\mathrm{i}}^{\mathrm{n}}((\mathrm{x_{ri}}-\mathrm{x_{mi}})^2+(\mathrm{y_{ri}}-\mathrm{y_{mi}})^2))}}{n} \ldots\ldots\ldots\ldots\ldots \text{(4.1)}$$

The Horizontal error at the point i $= \sum_{\mathrm{i}}^{\mathrm{n}}((\mathrm{x_{ri}} - \mathrm{x_{mi}})^2 + (\mathrm{y_{ri}} - \mathrm{y_{mi}})^2)$

or

The Horizontal error at the point i in x and y

=

$$\sigma_{\mathrm{x}}^2 = \sum_{i}^{n} \frac{(\mathrm{x_{ri}}-\mathrm{x_{mi}})^2}{n}$$

$$\sigma_{\mathrm{y}}^2 = \sum_{i}^{n} \frac{(\mathrm{y_{ri}} - \mathrm{y_{mi}})^2}{n}$$

Where:, x_r and y_r are the reference coordinates and x_m and y_m are the map or image coordinates for the ith sample point, $\sigma_{\mathrm{x}}^2, \sigma_{\mathrm{y}}^2$ are second momentum or variance of the statistical distribution.

The second momentum or variance is the square root of the average of the set of squared differences between map coordinates and their corresponding in-situ coordinates. Errors in X and Y coordinates are computed separately because they are independent of each other. In theory, it is assumed that errors in the spatial data have random behavior and that systematic errors have been eliminated as best as possible. Assuming that errors are normally distributed and independent of each other in the X and Y component, a factor of 2.4477 is used to compute horizontal accuracy at 95 % confidence level using equation 4.2 (NSSDA, 1989).

$$\text{Accuracy} = \frac{2.4477\sqrt{\sum_i^n \frac{((x_{ri}-x_{mi})^2+((y_{ri}-y_{mi})^2)}{n}}}{2} \text{............ (4.2)}$$

or

$$Accuracy=2.4477\frac{\sqrt{(\sigma_x^2+\sigma_y^2)}}{2} \quad \text{.. (4.3)}$$

If $\sigma_x^2 = \sigma_y^2$, equations (4.2) and (4.3) can be reduced to a more simplified formula. Let

RMSE h = $\sqrt{2}$ $*$RMSE$_X$ = $\sqrt{2}$ $*$RMSE$_Y$

Where

$$\text{RMSE}_x = \sqrt{\sigma_x^2}$$

$$\text{RMSE}_y = \sqrt{\sigma_y^2}$$

Or equivalently, the RMSE in x- and y-directions can be given as

$$\text{RMSE x} = \frac{\text{SQRT}(\Sigma(x_{ri}-x_{mi})^2)}{n}$$

$$\text{RMSE y}= \frac{\text{SQRT}(\Sigma\,(y_{ri}-y_{mi})^2)}{n}$$

Therefore, the horizontal accuracy can be given by:

$$Accuracy = 2.4477\frac{RMSE_h}{2} = 1.22385RMSE_h = 1.7308RMSE_x = 1.7308RMSE_y \quad \text{......... (4.4)}$$

Note that the constants 1.7308 and 1.22385 are the standard value from the x-axis of the standard normal distribution for an interval with a probability of 95%.

Note that this study only carried out horizontal accuracy assessment due to the fact that vertical coordinates are not available for the selected check points on orthophoto and digital line map.

CHAPTER FIVE

5. RESULT AND DISCUSSION

This thesis investigates the accuracy of orthophoto and its digital line map in horizontal position. Detail results from the accuracy assessment of orthophoto and digital line map are discussed in the next sections.

5.1 Positional Accuracy Assessment

To determine the coordinates of each checkpoints different methods of GNSS processing softwares packages were used. We need three remote geodetic GNSS processing services namely: OPUS (Online Positioning User Service) of the US National Geodetic Survey (NGS), AUSPOS in Australia[12], and APPS of the NASAJet Propulsion Laboratory[13]. OPUS uses baseline processing of standardized RINEX files into fixed positions. APPS, which is based onJet Propulsion Laboratory (http://apps.gdgps.net/). While AUSPOS, which is based on the Bernese software[14], and it allows the networked processing of observations from several user stations in RINEX format, using regional IGS stations for reference. APPS, uses GIPSY-OASIS (https://gipsy-oasis.jpl.nasa.gov/), makes use of a PPP-like approach (Precise Point Positioning). In APPS processing user stations are positioned one by one, using regional IGS stations. Results from this processing services are very similar in practice.But spectrum survey office software to create check points to conduct positional accuracy assessement of the given orthophoto. To process the GPS data of the twenty checkpoints with a tie to local GPS reference station avaliable over short baseline distance. In this instance, ADIS CORS station was used as a reference station and field observation on the ground was conducted by SOKKIA GRX1.

Prior to data processing datasets were collected from different stations,and also data from a number of CORS were downloaded. Firstly the station ADIS, at the Geophysical Observatory of Addis Ababa University, was used. It is equipped with a JPS Legacy E receiver with choke-ring antenna tracking both GPS and GLONASS satellites.

[12] (https://tinyurl.com/AUSPOSx)
[13] (http://apps.gdgps.net/)
[14] (http://www.bernese.unibe.ch/)

In the processing using remote geodetic GNSS processing services the OPUS, AUSPOS and APPS were used for processing of GNSS data with a tie to IGS reference stations ADIS IGS station was not on offered in AUSPOS and APPS but OPUS online processing get ADIS core station with other core stations.

5.2 GNSS Processing

Outputs obtained with the various remotely sensed online processing and GNSS software's are summarized in Table 5.1. It is seen that the differences between all the GNSS methods and software's are overall sub-centimeter as expected for geodetic GNSS with different occupation times. In describing our results, we have used the values obtained with the AUSPOS service as representative for all GNSS results. Compared to the co-ordinates read from the orthophoto map, they may be considered errorless.

Table 5.1-Deviation in Easting and Northing OPUS and Auspos with that of twenty check points

Deviation in Easting and Northing OPUS and Auspos								
Point number	Point description	x (OPUS)	x (AUSPOS)	diff in x	y (OPUS)	y (AUSPOS)	diff in y	$(\text{diff in x})^2 + (\text{diff in y})^2$
1		483915.47	483915.51	0.007	992911.52	992911.51	-0.037	0.0014
2		484508.14	484508.18	0.031	995014.72	995014.69	-0.048	0.0033
3		486206.67	486206.71	0.013	996892.43	996892.41	-0.036	0.0015
4		484108.51	484108.58	0.023	996914.79	996914.77	-0.07	0.0054
5		482634.58	482634.51	-0.02	995840.68	995840.7	0.072	0.0056
6		483353.49	483353.33	0.042	994867.07	994867.02	0.153	0.0252
7		481700.22	481700.37	-0.015	994464.42	994464.44	-0.15	0.0227
8		480970.78	480970.82	0.006	996901.29	996901.29	-0.033	0.0011
9		483215.06	483215.12	-0.005	996960.07	996960.07	-0.057	0.0033
10		482491.19	482491.22	0.004	998139.21	998139.2	-0.035	0.0012
11		480774.67	480774.74	0.014	997464.33	997464.32	-0.064	0.0043
12		480088.68	480088.68	0.001	995353.69	995353.69	0.002	0
13		480231.09	480231.09	-0.002	994566.05	994566.05	0.001	0
14		478091.3	478091.27	0.01	996971.15	996971.14	0.035	0.0013
15		478365.52	478365.47	0.046	995644.19	995644.15	0.047	0.0043
16		479761.96	479761.97	0.019	996840.02	996840	-0.013	0.0005
17		480360.3	480360.2	0	996032.75	996032.75	0.103	0.0106
18		478987.81	478987.85	0.002	996182.83	996182.83	-0.038	0.0014
19		481531.72	481531.72	-0.014	996407.32	996407.34	0.006	0.0002
20		483538.47	483538.45	0.038	998524.46	998524.43	0.026	0.0021
Mean =				**-0.01**			**-0.006**	
Std =				**0.06**			**0.067**	

Table 5.2-Deviation in Easting and Northing Sokkia vs Auspos with that of twenty check points

Deviation in Easting and Northing Sokkia vs Auspos								
Point number	Point description	X (SPECTRUM)	x (AUSPOS)	diff in x	y (SPECTRUM)	Y (AUSPOS)	diff in y	$(\text{diff in } x)^2 + (\text{diff in } y)^2$
1		483915.34	483915.51	-0.192	992911.32	992911.51	-0.164	0.06376
2		484508.25	484508.18	0.029	995014.72	995014.69	0.061	0.004562
3		486206.74	486206.71	0.026	996892.44	996892.41	0.038	0.00212
4		484108.64	484108.58	0.026	996914.8	996914.77	0.053	0.003485
5		482634.55	482634.51	0.026	995840.72	995840.7	0.043	0.002525
6		483353.65	483353.33	0.076	994867.1	994867.02	0.311	0.102497
7		481700.28	481700.37	0.003	994464.44	994464.44	-0.085	0.007234
8		480970.68	480970.82	-0.014	996901.27	996901.29	-0.137	0.018965
9		483215.16	483215.12	0.003	996960.07	996960.07	0.04	0.001609
10		482491.27	482491.22	0.014	998139.22	998139.2	0.046	0.002312
11		480774.78	480774.74	0.002	997464.32	997464.32	0.037	0.001373
12		480088.72	480088.68	0.025	995353.71	995353.69	0.033	0.001714
13		480231.13	480231.09	0.021	994566.07	994566.05	0.038	0.001885
14		478091.29	478091.27	0.033	996971.18	996971.14	0.02	0.001489
15		478365.5	478365.47	0.012	995644.16	995644.15	0.025	0.000769
16		479762.02	479761.97	0.035	996840.04	996840	0.049	0.003626
17		480360.19	480360.2	-0.023	996032.73	996032.75	-0.009	0.00061
18		478987.69	478987.85	-0.037	996182.79	996182.83	-0.152	0.024473
19		481531.56	481531.72	-0.03	996407.31	996407.34	-0.161	0.026821
20		483538.27	483538.45	-0.114	998524.31	998524.43	-0.174	0.043272
Mean =				**-0.004**			**-0.004**	
Std =				**0.058**			**0.114**	

Table 5.3 - Deviation in Easting and Northing Sokkia vs Opus with that of twenty check points

Deviation in Easting and Northing Sokkia vs Opus								
Point	Point	X	x		y	Y		$(\text{diff in x})^2$ +
number	description	(SPECTRUM)	(OPUS)	diff in x	(SPECTRUM)	(OPUS)	diff in y	$(\text{diff in y})^2$
1		483915.34	483915.47	-0.199	992911.32	992911.52	-0.127	0.05573
2		484508.25	484508.14	-0.002	995014.72	995014.72	0.109	0.011885
3		486206.74	486206.67	0.013	996892.44	996892.43	0.074	0.005645
4		484108.64	484108.51	0.003	996914.8	996914.79	0.123	0.015138
5		482634.55	482634.58	0.046	995840.72	995840.68	-0.029	0.002957
6		483353.65	483353.49	0.034	994867.1	994867.07	0.158	0.02612
7		481700.28	481700.22	0.018	994464.44	994464.42	0.065	0.004549
8		480970.68	480970.78	-0.02	996901.27	996901.29	-0.104	0.011216
9		483215.16	483215.06	0.008	996960.07	996960.07	0.097	0.009473
10		482491.27	482491.19	0.01	998139.22	998139.21	0.081	0.006661
11		480774.78	480774.67	-0.012	997464.32	997464.33	0.101	0.010345
12		480088.72	480088.68	0.024	995353.71	995353.69	0.031	0.001537
13		480231.13	480231.09	0.023	994566.07	994566.05	0.037	0.001898
14		478091.29	478091.3	0.023	996971.18	996971.15	-0.015	0.000754
15		478365.5	478365.52	-0.034	995644.16	995644.19	-0.022	0.00164
16		479762.02	479761.96	0.016	996840.04	996840.02	0.062	0.0041
17		480360.19	480360.3	-0.023	996032.73	996032.75	-0.112	0.013073
18		478987.69	478987.81	-0.039	996182.79	996182.83	-0.114	0.014517
19		481531.56	481531.72	-0.016	996407.31	996407.32	-0.167	0.028145
20		483538.27	483538.47	-0.152	998524.31	998524.46	-0.2	0.063104
mean =				**-0.013**			**0.002**	
std =				**0.06**			**0.106**	

5.3 Horizontal postiononal accuracy assessement

The RMSE of the difference between the orthophoto coordinates and in-situ GPS coordinates of the twenty check points in x and y-directions are **9.3** cm and 9.9 cm, respectively. The horizontal accuracy of orthophoto is determined at 3.19 cm with 95% confidence level (Table 5.4).

As indicated by Greenfeld (2001), the digital orthophoto was evaluated in the context of parcel based GIS with RMSE in x and y-direction (279 and 334 cm). In the same respect, Rabiu and Waziri (2014) evaluated the orthophoto and achieved RMSE of 18.0 cm in x-direction (RMSEx) and 22.6 cm in y-direction (RMSEy). Besides, ASPRS (2014) argued that spatial data accuracy standard should be within the error ranges in terms of RMSE in x, y and z-directions corresponding to the following values 10.2 cm, 10.6 cm, and 8.1 cm, respectively. Therefore, we can argued that this thesis showed progress in horizontal accuracy assessment, which approximately corresponds to an accuracy of 0.9 cm and 4.14 cm in orthophoto in x- and y-directions respectively, meeting national and international standards. In this study the RMSE in z-direction was not computed, because the topographic information was not available.

Table 5.4- Comparison of GPS data with that of Orthophoto

Comparison of GPS data with that of orthophoto								
Point number	Point description	x (AUSPOS)	x (ORTHO)	diff in x	Y (AUSPOS)	y (ORTHO)	diff in y	(diff in x)^2 + (diff in y)^2
1		483915.5	483916.006	-0.4978	992911.51	992911.525	-0.0153	0.248039
2		484508.2	484508.189	-0.0045	995014.69	995014.737	-0.0438	0.001939
3		486206.7	486206.427	0.279	996892.41	996892.338	0.0746	0.083406
4		484108.6	484108.577	0.0064	996914.77	996914.946	-0.1767	0.031264
5		482634.5	482634.611	-0.1041	995840.7	995840.739	-0.0411	0.012526
6		483353.3	483353.601	-0.2668	994867.02	994867.129	-0.1064	0.082503
7		481700.4	481700.387	-0.0197	994464.44	994464.525	-0.0899	0.00847
8		480970.8	480970.592	0.2243	996901.29	996901.149	0.1356	0.068698
9		483215.1	483215.208	-0.0874	996960.07	996960.122	-0.0507	0.010209
10		482491.2	482491.401	-0.1786	998139.2	998139.332	-0.1308	0.049007
11		480774.7	480774.578	0.16	997464.32	997464.359	-0.0398	0.027184
12		480088.7	480088.809	-0.1274	995353.69	995353.747	-0.0594	0.019759
13		480231.1	480230.953	0.1363	994566.05	994565.991	0.0578	0.021919
14		478091.3	478091.212	0.054	996971.14	996971.120	0.022	0.0034
15		478365.5	478365.605	-0.1328	995644.15	995643.928	0.2173	0.064855
16		479762	479762.008	-0.035	996840	996840.129	-0.125	0.01685
17		480360.2	480360.007	0.1923	996032.75	996032.747	0.0071	0.03703
18		478987.8	478988.239	-0.3927	996182.83	996182.193	0.6327	0.554523
19		481531.7	481531.797	-0.0795	996407.34	996407.327	0.0096	0.006412
20		483538.4	483538.402	0.0461	998524.43	998524.520	-0.0951	0.011169
mean =				**-0.0414**			**0.009**	
std =				**0.198**			**0.174**	
RMSE				**0.00**458			**0.0956**	

RMSEr= $= SQRT (RMSEx^2 + RMSEy^2)$

= 0.055

NSSDA Horizontal positional accuracy @ 95% confidence level = RMSEr * 1.7308 = 0.095 m

In addition to online data processing, the study used Spectrum survey software to establish checkpoints to conduct positional accuracy assessment of the given orthophoto because during creation of checkpoints in the study its uses Sokkia GRX1 instrument to collect datasets. So to process Sokkia data the best fitted processing software is Spectrum survey with full license. GPS data of the twenity checkpoints was processed with a tie to a local GPS reference station. In this case, ADIS CORS station was used as a reference station and field observation was conducted by SOKKIA GRX1.

The accuracy of orthophoto as compared to the GPS data acquired from Spectrum survey processing are 0.52 cm and 4.58 cm in x and y-direction, respectively in RMSE sense (Table 5.5). In other words, the positional accuracy of orthophoto evaluated for twenty checkpoints are estimated at 5.09 cm at 95 % confidence level. In this case, the maximum baseline distance from reference station to checkpoint location was 12 km.

Table 5.5- Comparison between GNSS (Sokkia) With Orthophoto

A Point number	B Point description	Deviation between GNSS(sokkia) with ortho					
		x (SPECTRUM)	x (ORTHO)	diff in x	y (SPECTRUM)	y (ORTHO)	diff in y
1		483915.344	483916.006	-0.6618	992911.318	992911.525	-0.2073
2		484508.245	484508.189	0.0565	995014.722	995014.737	-0.0148
3		486206.744	486206.427	0.317	996892.439	996892.338	0.1006
4		484108.636	484108.577	0.0594	996914.795	996914.946	-0.1507
5		482634.55	482634.611	-0.0611	995840.724	995840.739	-0.0151
6		483353.645	483353.601	0.0442	994867.099	994867.129	-0.0304
7		481700.282	481700.387	-0.1047	994464.438	994464.525	-0.0869
8		480970.679	480970.592	0.0873	996901.271	996901.149	0.1216
9		483215.161	483215.208	-0.0474	996960.074	996960.122	-0.0477
10		482491.268	482491.401	-0.1326	998139.215	998139.332	-0.1168
11		480774.775	480774.578	0.197	997464.321	997464.359	-0.0378
12		480088.715	480088.809	-0.0944	995353.713	995353.747	-0.0344
13		480231.127	480230.953	0.1743	994566.07	994565.991	0.0788
14		478091.286	478091.212	0.074	996971.175	996971.120	0.055
15		478365.497	478365.605	-0.1078	995644.157	995643.928	0.2293
16		479762.022	479762.008	0.014	996840.039	996840.129	-0.09
17		480360.19	480360.007	0.1833	996032.731	996032.747	-0.0159
18		478987.694	478988.239	-0.5447	996182.789	996182.193	0.5957
19		481531.556	481531.797	-0.2405	996407.307	996407.327	-0.0204
20		483538.274	483538.402	-0.1279	998524.311	998524.520	-0.2091
mean =				**0.0458**			**-0.00525**
std =				**0.2347**			**0.1757**
RMSE=				**0.2139**			**0.072**

RMSEr = $= SQRT\ (RMSEx^2 + RMSEy^2)$

$= 0.117$

NSSDA Horizontal positional accuracy @ 95% confidence level = RMSEr * 1.7308 = 0.2037 m

CHAPTER SIX

6. CONCLUSIONS AND RECOMMENDATIONS

The goal of this study was to conduct accuracy assessment of the digital orthophoto for large scale mapping production such as cadastral map. The major components were positional accuracy assessment in point wise. The positional accuracy (horizontally) was evaluated mainly in terms of $RMSE_h$ and 95 % confidence level and RMSE in x, y-direction from the two datasets (GPS survey checkpoints and orthophoto derived coordinates) and the standard deviation. Therefore, the study has drawn the following conclusions and recommendations.

6.1 Conclusions

From the results of this study, the following conclusions were made:

- The accuracy of the orthophoto compared to GPS data is within the national standard of error budget (~25cm) in terms of position. Therefore, asking the appropriateness of orthophoto for the support of large scale map production is the valid question. The question of appropriateness has to be answered by examining the specific intended usage of the source datasets (orthphoto and line map), which means a judgment call based on the intended use or application. From the result supported, the orthophoto is considered as appropriate for cadastral mapping applications, utility mapping, infrastructural mapping, road network mapping etc. Thus large scale map could be prepared from aerial surveying products such as (orthophoto).
- For this study different processing software and online processing was used and with this process the accuracy of each methods differ as compared each to them but their accuracy meet the standards set by national level (30cm). However, there are also some weaknesses in LGO, it’s difficult to process GNNS data imported from other instruments. The software needs compatible receiver types for baseline computation and it lacks to create link to precise earth rotation parameter, pole tide corrections and solid earth tide models.

6.2 Recommendations

As a way of improving beyond the scope of this study, I would like to recommend that future research be done in the following concepts:

- The accuracy of orthophoto can be checked by increasing GNSS observation time with this number of checkpoints.
- The accuracy of orthophoto can be further examined by incorporating online processing systems and processing software packages that are compatible with different GNNS receivers in order to make the robust comparative analysis.

-

REFERENCE

American Society for Photogrammetry and Remote Sensing (ASPRS), Accuracy Standards for Digital Geospatial Data, December, 2013.

David Siriba, Positional Accuracy Assessment of a Cadastral Dataset based on the Knowledge of the Process steps used Institute of Cartography and Geoinformatics, Leibniz Universität Hannover.2009

G.W. ChandrarathneComparative study on Cadastral Surveying using Total Station and High Resolution UAV Image.(HRUAVI),Institute of surveying and mapping (Diyatalawa) Survey Department of Sri Lanka JUNE, 2016.

Gakstatter, E., Silver, M.: "A Comparison of Free GPS Online Post-Processing Services", GPS World, http://gpsworld.com/a-comparison-of-free-gps- online-post-processing-services. (2013)

Geoffrey John Lawford, Analysis of the Positional Accuracy of Linear Features, Department of Geomatics, the University of Melbourne, September 2006

Habib, M. Morgan, E.M. Kim, R. Cheng, linear features in photogrammetric activities , Department of Geomatics Engineering, University of Calgary, Calgary

J. E. KING, Chief Photogrammetry zn Cadastral Surveying, Cartographic Section, Division Engineering, U. S. Forest Service.

Joon Kyu Park1, DaeWook Park, Application of the Ortho Image for the Cadastral Survey Dept. of Civil Engineering, Seoil University, Yongmasan-ro 90-gil, Jungnang-gu, Seoul, Korea

Lao-Sheng Lin, application of GPS RTK and Total station system on dynamic monitoring land use, Department of Land Economics, National Chengchi University, 64, Section 2, ChihnanRoad,Taipei 116, Taiwan, Republic of China.

Lawali and Waziri, digital orthophoto generation with aerial photographs, (2014)

National Standard for Spatial Data Accuracy, (NSSDA) Positional Accuracy Handbook (FGDC, 1999)

Ministry of Urban Development, Housing and Construction (2015). Urban Legal Cadastre Standard No. 03/2015. Technical report, Addis Ababa, Ethiopia.

ÖmerMutluoglu and AyhanCeylan, Accuracy and cost comparison of spatial data acquisition methods for the development of geographical information systems Selcuk University, Department of Geodesy and Photogrammetry, Konya, Turkey.July, 2009.

Peter Jenkins, LS, GPS/RTK Accuracy & Procedure Report Concerning Ground Control for Aerial Photography, 2006

Positional Accuracy Handbook, Using the National Standard for Spatial Data Accuracy to measure and report geographic data quality, OCTOBER 1999

Solomon Dargie Chekole, Surveying with GPS, total station and terrestrial laser scanner, School of Architecture and the Built Environment, Royal Institute of Technology (KTH),Stockholm, Sweden, May 2014.

Tina Dzigbordi Wemegah* Mark Brook man Amissah, Accuracy Checks in the Production of Orthophotos, Vol. 3, No.12, 2013, Department of Civil Engineering

World meteorological organization (WMO), www.wmo.int

Zinabu Getahun Sisaya, Tulu Beshab, Berhan Gessesse,feature orientation and positional accuracy assessment of digital orthophoto and line map for large scale mapping: the case study on bahirdar town, ethiopia ,Entoto Observatory and Research Center, 2017.

http://igs.bkg.bund.de/file/rinexsearch

https://www.ga.gov.au/scientific-topics/positioning-navigation/geodesy/auspos

https://www.gpsworld.com/a-comparison-of-free-gps-online-post-processing-services/

(https://tinyurl.com/AUSPOSx)

(http://apps.gdgps.net/)

(http://www.bernese.unibe.ch/)

ANNEX

FILE: log0207l.18O OP1535950434265

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: September 03, 2018
RINEX FILE: log0207l.18o TIME: 04:54:58 UTC

SOFTWARE: page5 1603.24 master93.pl 160321 START: 2018/07/26 11:24:00
EPHEMERIS: igs20114.eph [precise] STOP: 2018/07/26 13:27:00
NAV FILE: brdc2070.18n OBS USED: 5721 / 5942 : 96%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 33 / 38 : 87%
ARP HEIGHT: 1.514 OVERALL RMS: 0.022(m)

REF FRAME: IGS08 (EPOCH:2018.5658)

X: 4908161.763(m) 0.019(m)
Y: 3953958.190(m) 0.131(m)
Z: 989807.817(m) 0.048(m)

LAT: 8 59 3.45610 0.035(m)
E LON: 38 51 16.25630 0.102(m)
W LON: 321 8 43.74370 0.102(m)
EL HGT: 2314.174(m) 0.088(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 993118.866
Easting (X) [meters] 484008.528
Convergence [degrees] -0.02271944
Point Scale 0.99960316
Combined Factor 0.99923944

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
BHR4				2263218.9
ADIS				11216.3
MBAR				1388226.9

FILE: log0208h.18O OP1534253374240

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0208h.18o TIME: 13:32:10 UTC

SOFTWARE: page5 1603.24 master93.pl 160321 START: 2018/07/27 07:24:00
EPHEMERIS: igs20115.eph [precise] STOP: 2018/07/27 09:26:00
NAV FILE: brdc2080.18n OBS USED: 4230 / 4568 : 93%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 28 / 36 : 78%
ARP HEIGHT: 1.550 OVERALL RMS: 0.019(m)

REF FRAME: IGS08 (EPOCH:2018.5681)

X: 4906298.479(m) 0.013(m)
Y: 3955399.569(m) 0.058(m)
Z: 993755.771(m) 0.035(m)

LAT: 9 1 13.11085 0.042(m)
E LON: 38 52 31.25179 0.037(m)
W LON: 321 7 28.74821 0.037(m)
EL HGT: 2392.299(m) 0.041(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 997099.774
Easting (X) [meters] 486299.726
Convergence [degrees] -0.01954444
Point Scale 0.99960232
Combined Factor 0.99922633

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ADIS			12105.4
	MAL2			1334593.4
	BHR4			2258712.4

FILE: log0208h.18O OP1534251302903

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0208h.18o TIME: 12:57:38 UTC

SOFTWARE: page5 1603.24 master93.pl 160321 START: 2018/07/27 07:53:00
EPHEMERIS: igs20115.eph [precise] STOP: 2018/07/27 09:56:00
NAV FILE: brdc2080.18n OBS USED: 3594 / 4102 : 88%

ANT NAME: SOKGRX1 NONE # FIXED AMB: 21 / 35 : 60%
ARP HEIGHT: 1.425 OVERALL RMS: 0.015(m)

REF FRAME: IGS08 (EPOCH:2018.5681)

X: 4907552.770(m) 0.050(m)
Y: 3954228.154(m) 0.033(m)
Z: 991890.814(m) 0.031(m)

LAT: 9 0 11.94872 0.028(m)
E LON: 38 51 35.64054 0.057(m)
W LON: 321 8 24.35946 0.057(m)
EL HGT: 2338.692(m) 0.024(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 995222.073
Easting (X) [meters] 484601.193
Convergence [degrees] -0.02192500
Point Scale 0.99960293
Combined Factor 0.99923536

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE DISTANCE(m)
	BHR4		2261154.1
	ADIS		10880.5
	MBAR		1390199.3

FILE: log0208l.18O OP1534251249018

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0208l.18o TIME: 12:56:43 UTC

SOFTWARE: page5 1603.24 master94.pl 160321 START: 2018/07/27 11:43:00
EPHEMERIS: igs20115.eph [precise] STOP: 2018/07/27 13:46:00
NAV FILE: brdc2080.18n OBS USED: 5212 / 5588 : 93%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 36 / 49 : 73%
ARP HEIGHT: 1.535 OVERALL RMS: 0.021(m)

REF FRAME: IGS08 (EPOCH:2018.5686)

X: 4907609.212(m) 0.117(m)
Y: 3953759.098(m) 0.083(m)

Z: 993776.251(m) 0.057(m)

LAT: 9 1 13.81404 0.068(m)
E LON: 38 51 22.52722 0.058(m)
W LON: 321 8 37.47278 0.058(m)
EL HGT: 2386.786(m) 0.135(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 997122.140
Easting (X) [meters] 484201.570
Convergence [degrees] -0.02253611
Point Scale 0.99960309
Combined Factor 0.99922797

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ADIS			10025.6
	MAL2			1334846.4
	BHR4			2259776.4

FILE: log0208m.18O OP1534253340375

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0208m.18o TIME: 13:31:46 UTC

SOFTWARE: page5 1603.24 master54.pl 160321 START: 2018/07/27 12:11:00
EPHEMERIS: igs20115.eph [precise] STOP: 2018/07/27 14:13:00
NAV FILE: brdc2080.18n OBS USED: 5365 / 5879 : 91%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 40 / 63 : 63%
ARP HEIGHT: 1.526 OVERALL RMS: 0.021(m)

REF FRAME: IGS08 (EPOCH:2018.5686)

X: 4908628.802(m) 0.009(m)
Y: 3952686.857(m) 0.263(m)
Z: 992706.521(m) 0.019(m)

LAT: 9 0 38.81880 0.031(m)
E LON: 38 50 34.26421 0.201(m)
W LON: 321 9 25.73579 0.201(m)
EL HGT: 2339.114(m) 0.167(m)

UTM COORDINATES

UTM (Zone 37)
Northing (Y) [meters] 996048.026
Easting (X) [meters] 482727.637
Convergence [degrees] -0.02461389
Point Scale 0.99960369
Combined Factor 0.99923606

BASE STATIONS USED
PID DESIGNATION LATITUDE LONGITUDE DISTANCE(m)
ADIS 8840.7
MAL2 1333943.9
MBAR 1389607.1
FILE: log0209h.18O OP1534250741122

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0209h.18o TIME: 12:48:33 UTC

SOFTWARE: page5 1603.24 master75.pl 160321 START: 2018/07/28 07:29:00
EPHEMERIS: igs20116.eph [precise] STOP: 2018/07/28 09:31:00
NAV FILE: brdc2090.18n OBS USED: 3619 / 3962 : 91%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 39 / 46 : 85%
ARP HEIGHT: 1.464 OVERALL RMS: 0.020(m)

REF FRAME: IGS08 (EPOCH:2018.5708)

X: 4908305.452(m) 0.172(m)
Y: 3953350.736(m) 0.217(m)
Z: 991746.390(m) 0.026(m)

LAT: 9 0 7.12570 0.022(m)
E LON: 38 50 57.82423 0.260(m)
W LON: 321 9 2.17577 0.260(m)
EL HGT: 2351.351(m) 0.057(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 995074.414
Easting (X) [meters] 483446.544
Convergence [degrees] -0.02356389
Point Scale 0.99960339
Combined Factor 0.99923383

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE DISTANCE(m)
	ADIS		9848.7
	MBAR		1389340.2
	BHR4		2261876.6

FILE: log0209i.18O OP1534253293647

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0209i.18o TIME: 13:30:42 UTC

SOFTWARE: page5 1603.24 master95.pl 160321 START: 2018/07/28 08:18:00
EPHEMERIS: igs20116.eph [precise] STOP: 2018/07/28 10:19:00
NAV FILE: brdc2090.18n OBS USED: 4378 / 4933 : 89%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 29 / 44 : 66%
ARP HEIGHT: 1.439 OVERALL RMS: 0.015(m)

REF FRAME: IGS08 (EPOCH:2018.5709)

X: 4909343.937(m) 0.006(m)
Y: 3952062.936(m) 0.020(m)
Z: 991337.879(m) 0.029(m)

LAT: 8 59 53.99146 0.028(m)
E LON: 38 50 3.68077 0.016(m)
W LON: 321 9 56.31923 0.016(m)
EL HGT: 2288.612(m) 0.015(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 994671.769
Easting (X) [meters] 481793.275
Convergence [degrees] -0.02590833
Point Scale 0.99960410
Combined Factor 0.99924440

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE DISTANCE(m)
	BHR4		2263057.2
	ADIS		8522.5
	MBAR		1387957.5

FILE: log0209m.18O OP1534253250070

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0209m.18o TIME: 13:30:13 UTC

SOFTWARE: page5 1603.24 master91.pl 160321 START: 2018/07/28 12:23:00
EPHEMERIS: igs20116.eph [precise] STOP: 2018/07/28 14:25:00
NAV FILE: brdc2090.18n OBS USED: 5429 / 5675 : 96%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 33 / 42 : 79%
ARP HEIGHT: 1.556 OVERALL RMS: 0.023(m)

REF FRAME: IGS08 (EPOCH:2018.5714)

X: 4908159.979(m) 0.048(m)
Y: 3953054.583(m) 0.055(m)
Z: 993819.693(m) 0.019(m)

LAT: 9 1 15.27650 0.016(m)
E LON: 38 50 53.26202 0.059(m)
W LON: 321 9 6.73798 0.059(m)
EL HGT: 2380.726(m) 0.050(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 997167.414
Easting (X) [meters] 483308.121
Convergence [degrees] -0.02381389
Point Scale 0.99960345
Combined Factor 0.99922928

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE DISTANCE(m)
	ADIS	9136.6	
	BHR4	2260199.3	
	MBAR	1390832.6	

FILE: log0209m.18O OP1534250658582

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0209m.18o TIME: 12:47:15 UTC

SOFTWARE: page5 1603.24 master58.pl 160321 START: 2018/07/28 12:56:00
EPHEMERIS: igs20116.eph [precise] STOP: 2018/07/28 14:59:00
NAV FILE: brdc2090.18n OBS USED: 5010 / 5543 : 90%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 33 / 49 : 67%
ARP HEIGHT: 1.468 OVERALL RMS: 0.025(m)

REF FRAME: IGS08 (EPOCH:2018.5715)

X: 4909569.372(m) 0.133(m)
Y: 3951305.860(m) 0.098(m)
Z: 993759.309(m) 0.046(m)

LAT: 9 1 13.33024 0.045(m)
E LON: 38 49 39.75236 0.121(m)
W LON: 321 10 20.24764 0.121(m)
EL HGT: 2372.394(m) 0.099(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 997108.639
Easting (X) [meters] 481063.841
Convergence [degrees] -0.02701389
Point Scale 0.99960444
Combined Factor 0.99923158

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	MAL2			1335183.7
	MBAR			1389334.3
	BHR4			2261408.2

FILE: log0210i.18O OP1534252125105

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0210i.18o TIME: 13:11:34 UTC

SOFTWARE: page5 1603.24 master70.pl 160321 START: 2018/07/29 08:54:00
EPHEMERIS: igr20120.eph [rapid] STOP: 2018/07/29 10:57:00
NAV FILE: brdc2100.18n OBS USED: 5057 / 5288 : 96%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 30 / 41 : 73%
ARP HEIGHT: 1.637 OVERALL RMS: 0.018(m)

REF FRAME: IGS08 (EPOCH:2018.5737)

X: 4908500.814(m) 0.062(m)
Y: 3952398.280(m) 0.084(m)
Z: 994991.008(m) 0.014(m)

LAT: 9 1 53.66179 0.026(m)
E LON: 38 50 29.53496 0.027(m)
W LON: 321 9 30.46504 0.027(m)
EL HGT: 2420.116(m) 0.099(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 998346.553
Easting (X) [meters] 482584.245
Convergence [degrees] -0.02487500
Point Scale 0.99960375
Combined Factor 0.99922339

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE DISTANCE(m)
	ADIS		8284.1
	MAL2		1336241.2
	MBAR		1391258.6

FILE: log0210j.18O OP1534250495743

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0210j.18o TIME: 12:44:20 UTC

SOFTWARE: page5 1603.24 master92.pl 160321 START: 2018/07/29 09:18:00
EPHEMERIS: igr20120.eph [rapid] STOP: 2018/07/29 11:20:00
NAV FILE: brdc2100.18n OBS USED: 4691 / 4798 : 98%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 24 / 29 : 83%
ARP HEIGHT: 1.475 OVERALL RMS: 0.018(m)

REF FRAME: IGS08 (EPOCH:2018.5738)

X: 4909635.669(m) 0.030(m)
Y: 3951106.948(m) 0.085(m)
Z: 994318.150(m) 0.044(m)

LAT: 9 1 31.66108 0.046(m)
E LON: 38 49 33.32010 0.062(m)
W LON: 321 10 26.67990 0.062(m)

EL HGT: 2387.878(m)   0.054(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters]   997671.681
Easting (X) [meters]   480867.732
Convergence [degrees]   -0.02730833
Point Scale   0.99960453
Combined Factor   0.99922924

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ADIS			6644.3
	BHR4			2261041.1
	MBAR			1389634.6

FILE: log0211g.18O OP1534252017690

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com   DATE: August 14, 2018
RINEX FILE: log0211g.18o   TIME: 13:09:32 UTC

SOFTWARE: page5 1603.24 master74.pl 160321   START: 2018/07/30 06:14:00
EPHEMERIS: igr20121.eph [rapid]   STOP: 2018/07/30 08:16:00
NAV FILE: brdc2110.18n   OBS USED: 3621 / 4024   : 90%
ANT NAME: SOKGRX1   NONE   # FIXED AMB: 20 /   24   : 83%
ARP HEIGHT: 1.547   OVERALL RMS: 0.016(m)

REF FRAME: IGS08 (EPOCH:2018.5762)

X: 4910274.183(m)   0.047(m)
Y: 3950924.296(m)   0.096(m)
Z: 991441.859(m)   0.009(m)

LAT: 8 59 57.27815   0.013(m)
E LON: 38 49 15.56144   0.091(m)
W LON: 321 10 44.43856   0.091(m)
EL HGT: 2315.533(m)   0.039(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters]   994773.396
Easting (X) [meters]   480324.148
Convergence [degrees]   -0.02800000
Point Scale   0.99960479

Combined Factor 0.99924086

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
MAL2		1332953.4		
ADIS		7208.6		
BHR4		2263736.8		

FILE: log0211g.18O OP1534250370177

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0211g.18o TIME: 12:42:09 UTC

SOFTWARE: page5 1603.24 master72.pl 160321 START: 2018/07/30 06:41:00
EPHEMERIS: igr20121.eph [rapid] STOP: 2018/07/30 08:42:00
NAV FILE: brdc2110.18n OBS USED: 3739 / 4145 : 90%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 29 / 38 : 76%
ARP HEIGHT: 1.703 OVERALL RMS: 0.020(m)

REF FRAME: IGS08 (EPOCH:2018.5762)

X: 4910293.827(m) 0.028(m)
Y: 3950756.687(m) 0.070(m)
Z: 992225.641(m) 0.086(m)

LAT: 9 0 22.92318 0.083(m)
E LON: 38 49 10.88458 0.039(m)
W LON: 321 10 49.11542 0.039(m)
EL HGT: 2349.521(m) 0.075(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 995561.038
Easting (X) [meters] 480181.742
Convergence [degrees] -0.02822778
Point Scale 0.99960486
Combined Factor 0.99923559

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
BHR4		2263156.9		
MAL2		1333751.8		
ADIS		6679.8		

FILE: log0211k.18O OP1534251911796

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0211k.18o TIME: 13:07:45 UTC

SOFTWARE: page5 1603.24 master96.pl 160321 START: 2018/07/30 10:06:00
EPHEMERIS: igr20121.eph [rapid] STOP: 2018/07/30 12:09:00
NAV FILE: brdc2110.18n OBS USED: 3875 / 4766 : 81%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 38 / 54 : 70%
ARP HEIGHT: 1.520 OVERALL RMS: 0.023(m)

REF FRAME: IGS08 (EPOCH:2018.5766)

X: 4911337.324(m) 0.223(m)
Y: 3949382.728(m) 0.419(m)
Z: 992511.481(m) 0.198(m)

LAT: 9 0 32.35370 0.128(m)
E LON: 38 48 14.44021 0.187(m)
W LON: 321 11 45.55979 0.187(m)
EL HGT: 2346.819(m) 0.462(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 995851.540
Easting (X) [meters] 478458.578
Convergence [degrees] -0.03068889
Point Scale 0.99960574
Combined Factor 0.99923690

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
BHR4				2263805.7
MAL2				1334236.4
ADIS				5057.6

FILE: log0211k.18O OP1534249842343

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018

RINEX FILE: log0211k.18o TIME: 12:33:34 UTC

SOFTWARE: page5 1603.24 master72.pl 160321 START: 2018/07/30 10:30:00
EPHEMERIS: igr20121.eph [rapid] STOP: 2018/07/30 12:31:00
NAV FILE: brdc2110.18n OBS USED: 4684 / 5062 : 93%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 27 / 34 : 79%
ARP HEIGHT: 1.517 OVERALL RMS: 0.014(m)

REF FRAME: IGS08 (EPOCH:2018.5767)

X: 4911376.489(m) 0.027(m)
Y: 3949061.158(m) 0.184(m)
Z: 993828.792(m) 0.104(m)

LAT: 9 1 15.55752 0.095(m)
E LON: 38 48 5.43503 0.156(m)
W LON: 321 11 54.56497 0.156(m)
EL HGT: 2384.357(m) 0.110(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 997178.500
Easting (X) [meters] 478184.360
Convergence [degrees] -0.03112222
Point Scale 0.99960589
Combined Factor 0.99923115

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	BHR4			2262843.9
	MAL2			1335583.5
	ADIS			4176.9

FILE: log0211n.18O OP1534249555086

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0211n.18o TIME: 12:28:37 UTC

SOFTWARE: page5 1603.24 master73.pl 160321 START: 2018/07/30 13:09:00
EPHEMERIS: igr20121.eph [rapid] STOP: 2018/07/30 15:12:00
NAV FILE: brdc2110.18n OBS USED: 4563 / 5299 : 86%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 35 / 45 : 78%
ARP HEIGHT: 1.556 OVERALL RMS: 0.024(m)

REF FRAME: IGS08 (EPOCH:2018.5770)

X: 4910335.743(m) 0.117(m)
Y: 3950369.802(m) 0.202(m)
Z: 993698.259(m) 0.052(m)

LAT: 9 1 11.31607 0.019(m)
E LON: 38 49 0.15892 0.083(m)
W LON: 321 10 59.84108 0.083(m)
EL HGT: 2372.952(m) 0.223(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 997047.371
Easting (X) [meters] 479855.019
Convergence [degrees] -0.02873611
Point Scale 0.99960502
Combined Factor 0.99923207

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ADIS	5798.1		
	BHR4	2262085.8		
	MBAR	1388506.2		

FILE: log0212i.18O OP1534251565480

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0212i.18o TIME: 13:02:25 UTC

SOFTWARE: page5 1603.24 master58.pl 160321 START: 2018/07/31 08:47:00
EPHEMERIS: igr20122.eph [rapid] STOP: 2018/07/31 10:50:00
NAV FILE: brdc2120.18n OBS USED: 4571 / 5120 : 89%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 32 / 47 : 68%
ARP HEIGHT: 1.595 OVERALL RMS: 0.017(m)

REF FRAME: IGS08 (EPOCH:2018.5792)

X: 4910882.634(m) 0.094(m)
Y: 3949815.863(m) 0.025(m)
Z: 993044.455(m) 0.041(m)

LAT: 9 0 49.90352 0.028(m)
E LON: 38 48 34.81319 0.039(m)
W LON: 321 11 25.18681 0.039(m)
EL HGT: 2348.443(m) 0.094(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 996390.176
Easting (X) [meters] 479080.867
Convergence [degrees] -0.02981944
Point Scale 0.99960542
Combined Factor 0.99923632

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
BHR4		2263032.7		
ADIS		5319.7		
MAL2		1334697.3		

FILE: log0212i.18O OP1534249023554
1009 WARNING! No antenna type was selected. No antenna offsets or
1009 pattern will be applied. Coordinates with reduced accuracy
1009 will be returned for the antenna phase center.
1009

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0212i.18o TIME: 12:19:42 UTC

SOFTWARE: page5 1603.24 master72.pl 160321 START: 2018/07/31 08:12:00
EPHEMERIS: igr20122.eph [rapid] STOP: 2018/07/31 10:14:00
NAV FILE: brdc2120.18n OBS USED: 3396 / 3828 : 89%
ANT NAME: NONE NONE # FIXED AMB: 22 / 35 : 63%
ARP HEIGHT: 1.648 OVERALL RMS: 0.019(m)

REF FRAME: IGS08 (EPOCH:2018.5791)

X: 4910050.102(m) 0.159(m)
Y: 3950909.047(m) 0.083(m)
Z: 992898.886(m) 0.091(m)

LAT: 9 0 45.03931 0.077(m)
E LON: 38 49 19.77017 0.035(m)
W LON: 321 10 40.22983 0.035(m)
EL HGT: 2361.738(m) 0.187(m)

UTM COORDINATES
UTM (Zone 37)

Northing (Y) [meters] 996240.102
Easting (X) [meters] 480453.360
Convergence [degrees] -0.02785833
Point Scale 0.99960473
Combined Factor 0.99923354

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE (m)
	ADIS			6634.3
	BHR4			2262449.8
	MAL2			1334393.3

FILE: log0212n.18O OP1534247282900

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0212n.18o TIME: 11:50:47 UTC

SOFTWARE: page5 1603.24 master58.pl 160321 START: 2018/07/31 13:13:00
EPHEMERIS: igr20122.eph [rapid] STOP: 2018/07/31 15:11:00
NAV FILE: brdc2120.18n OBS USED: 5488 / 5718 : 96%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 27 / 38 : 71%
ARP HEIGHT: 1.710 OVERALL RMS: 0.018(m)

REF FRAME: IGS08 (EPOCH:2018.5797)

X: 4909269.952(m) 0.041(m)
Y: 3951785.776(m) 0.068(m)
Z: 993269.777(m) 0.047(m)

LAT: 9 0 57.25408 0.049(m)
E LON: 38 49 58.13310 0.043(m)
W LON: 321 10 1.86690 0.043(m)
EL HGT: 2362.490(m) 0.060(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 996614.671
Easting (X) [meters] 481624.781
Convergence [degrees] -0.02620000
Point Scale 0.99960418
Combined Factor 0.99923287

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	MAL2			1334630.8

MBAR 1389323.1
ADIS 7619.0
FILE: log0213i.18O OP1534246651283

NGS OPUS SOLUTION REPORT
========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: sisay2020@gmail.com DATE: August 14, 2018
RINEX FILE: log0213i.18o TIME: 11:40:10 UTC

SOFTWARE: page5 1603.24 master92.pl 160321 START: 2018/08/01 08:48:00
EPHEMERIS: igr20123.eph [rapid] STOP: 2018/08/01 10:49:00
NAV FILE: brdc2130.18n OBS USED: 4146 / 4653 : 89%
ANT NAME: SOKGRX1 NONE # FIXED AMB: 30 / 43 : 70%
ARP HEIGHT: 1.616 OVERALL RMS: 0.021(m)

REF FRAME: IGS08 (EPOCH:2018.5819)

X: 4907824.430(m) 0.120(m)
Y: 3953199.080(m) 0.122(m)
Z: 995377.961(m) 0.051(m)

LAT: 9 2 6.22097 0.068(m)
E LON: 38 51 3.83429 0.117(m)
W LON: 321 8 56.16571 0.117(m)
EL HGT: 2456.675(m) 0.123(m)

UTM COORDINATES
UTM (Zone 37)
Northing (Y) [meters] 998731.810
Easting (X) [meters] 483631.531
Convergence [degrees] -0.02338889
Point Scale 0.99960332
Combined Factor 0.99921722

BASE STATIONS USED
PID DESIGNATION LATITUDE LONGITUDE DISTANCE(m)
MBAR 1392231.4
BHR4 2258734.8
ADIS 9322.6