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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEODESY AND GEOMATICS PROGRAM

ASSESSMENT OF HORIZONTAL POSITIONAL ACCURACY OF DIGITAL ORTHOPHOTO IN THE TOWN OF CHIRO, ETHIOPIA.

BY:

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DECLARATION

I, the undersigned, declare that the thesis titled ‘‘Assessment of horizontal positional accuracy of digital orthophoto in the town of Chiro, Ethiopia’’ carried out under the supervision of **Dr. Tullu Besha Bedada** is my own work and has not been presented for a degree in any other university and that all sources of material used for the thesis have been appropriately acknowledged.

<u>Muhammed Siraj Sualih</u> ____________________ ______________________

Name of Candidate Signature Date

As Master research advisor, I hereby certify that I have read and evaluated this MSc thesis prepared under my guidance.

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The undersigned have examined the Thesis entitled ***''Assessment of Horizontal Positional Accuracy of Digital Orthophoto in the Town of Chiro, Ethiopia''*** Presented by **Muhammed Siraj Sualih,** a candidate for the Degree of Master of Science in Geodesy and Geomatics and herby certify it is worthy of acceptance.

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ABSTRACT

Positional accuracy is a matter of renewed concern and it deals with the accurateness of the location of map features, and measures how far a spatial feature on a map is from its true or reference location on the ground. A crucial component to examine an effective positional correctness of digital spatial data needs acquisition of better quality of independent ground reference data. This data makes spatial analysis less troublesome and enables better decision making. The objective of this thesis is to examine the horizontal positional accuracy of digital orthophoto. Accordingly, the study used in-situ Global Navigation Satellite System (GNSS) data to validate the accuracy of horizontal coordinates an Orthophoto located in Chiro town. GNSS data is observed using Leica GS 14 Differential Global Positioning System (GPS) instrument and post processed with different processing software's like Leica Geo-Office (LGO) and online processing software's using AUSPOS and OPUS with a tie to local and regional GNSS reference stations to obtain horizontal coordinates. In this paper the accuracy of horizontal coordinates of Digital Orthophoto was evaluated by comparing with the corresponding coordinates measured directly from in-situ GNSS data. As a result, the root mean square error between the orthophoto coordinates and in-situ GNSS coordinates of the nine check points in x and y-directions are 0.993 m and 1.462 m, 0.718 m and 1.712 m, 0.715 m and 1.709 m as compared to the GNSS data obtained from LGO, AUSPOS and OPUS processing respectively. In addition, the orthophoto horizontal accuracy was found to be 3.059 m, 3.213 m, and 3.206 m at 95 percent confidence level as compared to the GNSS data obtained from LGO, AUSPOS and OPUS processing respectively. Therefore, it can be concluded that, in all condition the accuracy of orthophoto doesn't meet the national standard of error budget (30cm) for the purpose of Standard Mapping work use. Rather, it can use for Visualization and less accurate work use. In its present state, it can be used for a Master plan and General Economic development plan application.

Keywords*:* GPS, GNSS, LGO, AUPOS, OPUS, Digital Orthophoto, Standard Mapping, Visualization, Master plan, General Economic Development plan.

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ACRONYMS

2D	Two Dimensional
3D	Three Dimensional
ASPRS	American Society of Photogrammetry and Remote Sensing
AUSPOS	Australia's On-Line Static GPS Positioning Service
CORS	Continuously Operating Reference Station
CSA	Central Statics Agency
DTM	Digital Terrain Model
EGII	Ethiopia Geospatial Information Institute
FIG	International Federation of Survey
FULLPRIA	Federal Urban Land and Land Related Property Registry and Information Agency
GCP	Ground Control Point
GDB	Geospatial Database
GDOP	Geometric Dilution of Precision
GIS	Geographic Information System
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
GSD	Ground Sample Distance
IGS	International GNSS Service
INSA	Information Network Security Agency
ISO	International Organization for Standardization
MAX	Maximum

MIN	Minimum
NMAs	National Mapping Agencies
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
UTM	Universal Transverse Mercator
WGS	World Geodetic System

CHAPTER ONE

1 INTRODUCTION

1.1 Background

An orthophoto is a computer-compatible graphic version in which a photograph showing imageries of an entities in their true orthographic positions. Therefore, it's geometrically equivalent to conventional line and symbol planimetric maps (Wolf et al., 2014). It has been used widely in several applications this time in an attempt to map the earth's surface in a shortest possible time and provide information for monitoring and planning (Wemegah and Amissah, 2013). Currently the Geological Survey is exploring the utility of large scale orthophoto products as a base map component of a local multipurpose cadastre for city, county, and regional agencies (Hood et al., 1989). Especially in the urban areas, it is about fast and relative cheap way to provide up-to-date spatial information than traditional cartographic techniques. Different procedures for quantification and monitoring of these changes in urban areas, based on digital orthophoto (Igor et al., 2004).

Although orthophoto generally offer significant benefits, not all orthophoto are created equal. There is different contributing error factors comprises the characteristics and calibration of equipment used during image acquirement for instance the camera and/or scanner. Additional elements which affects the rectification system include ground control points accuracy, the aerial triangulation process, digital elevation models (DEM) and the software used for rectification **(**Greenfeld, 2001). High number of Orthophoto usages in prior stages of most engineering plans which have caused to have more attention to both of the geometric and quality aspects. The error of aerial triangulation (AT) due to some obvious effects on orthophoto like as displacements (Modiri et al., 2015). Besides, the problem of spatial data quality is introduced and disseminated through the data acquisition, data processing and application. Positional accuracy, attribute accuracy, logical uniformity, integrity and correctness, time accuracy and semantic accuracy are used to describe the quality of spatial data. The consideration of aerial photography for particular mapping applications depends on the scale of photography. Traditionally, the positional and elevation accuracies attainable using photogrammetric approaches and the scale of the final map were decide based on mapping requirements and practice (Hood et al., 1989).

Positional accuracy is one element of spatial data quality and it deals about the correctness of the location of features within a spatial reference system (Drobnjak et al., 2017). And also, it's a matter of renewed concern because of the capabilities offered by the global positioning system (GPS) (López and Gordo, 2008). To be specific, horizontal positional accuracy is the exactness of the horizontal locations of the spatial entities which can be measured in terms of latitude and longitude or local easting and northing (ASPRS, 2014). Because of its crucial component of positional accurateness in cartographic production, all national mapping agencies (NMAs) have used statistical approaches for its control and the process termed positional accuracy assessment methodologies (López and Gordo, 2008).
Therefore, horizontal accurateness of spatial data can be evaluated using root mean square error (RMSE) statistics in the horizontal plane, i.e., RMSEx, RMSEy and RMSEh (ASPRS, 2014). In the end, the objective of this study is to examine the accuracy and quality of orthoimages with respect to the ground observation data.

1.2 Statement of the Problem

In developing countries the development of urban cadastral system supports the provision of security of tenure by way of ensuring that the information on urban tenure is formally acknowledged and using the information as a basis for any planning decisions and intrusions (Chekole et al., 2020). And also, photogrammetric methods considered as an alternative to ground based surveys to identify land parcel boundaries and it has been adopted in different ways by various countries at different periods of time (Siriba, 2009). Accordingly, Ethiopia has started urban cadastral mapping by using aerial photo to assure tenure security of peoples and improve land management system. Chiro town is one of the selected sites and an orthophoto has been produced to do so. In spite of this, orthophoto accuracy and quality will vary based on the data used to process them, as well as on the particular production procedure. Characteristics and calibration of equipment used for image capture such as the camera and/or scanner are the factor which makes effect on its correctness. In addition, the rectification and aerial triangulation process as well as the method or software used for rectification also affects the orthophoto quality. For this reason, it is essential to note that orthophoto should be evaluated for spatial accuracy and inspected for image quality before acceptance or use **(**Greenfeld, 2001). In the same manner, (Siriba, 2009) stated that to use these orthophoto images as a map, it is of great importance to provide orthophoto images in high level of accuracy. The accuracy of orthophoto and cadastral data has to be examined using in-situ ground measurements to realize the production of large scale maps (ranging from 1:500 to 1:10,000) based on orthophoto that can be reliably used for practical cadastral applications. Therefore, examining the quality of an orthophoto becomes a crucial component before to use for intended purpose. In addition to this only few studies have been conducted in Ethiopia to investigate the positional accuracy of digital orthophoto. Consequently, this study assessed the horizontal positional accuracy of orthophoto for Chiro Town.

1.3 Objectives

1.3.1 General Objective

The general objective of this study is intended to evaluate the horizontal positional accuracy of digital orthophoto of Chiro Town.

1.3.2 Specific Objectives

- To evaluate the results of GNSS derived coordinates obtained from different processing techniques.
- To assess the accuracy of horizontal coordinates of an orthophoto with in-situ GNSS measurements.

1.4 Significance of the Study

The study offers valuable information regarding the accuracy issues of digital orthophoto for the purpose of large scale mapping and other related applications. Therefore, this study will help to forthcoming researchers, government bodies, NGO and other concerned bodies who have an interest to pursue in related aspects of the research. In addition to this, since coordinates of the reference points are determined with high precision, it can serve as a reference value for intended use.

1.5 Thesis Structure

This thesis has five chapters. The first chapter introduces the overall background, problem statement, objective of the thesis and significance of the study. Whereas, chapter two has incorporated literature review, which describes the overview of digital orthophoto, positional accuracy assessment and it also presents other’s related work. In addition to this, chapter three describes about the study area, discusses with the data source, material and methodology. The fourth chapter consists of the results and discussion. Finally, conclusions and recommendations are presented in chapter five.

CHAPTER TWO

2 LITERATURE REVIEW

2.1 Introduction

A review of the existing literature associated in evaluating the horizontal position accurateness of digital orthophoto has provided insight into key investigation implemented on this area. An orthophoto is a computer-compatible pictorial rendition in which a photograph showing images of objects in their true orthographic positions. Therefore, it is geometrically equivalent to conventional line and symbol planimetric maps which also show true orthographic positions of objects (Wolf et al., 2014). Nowadays, a digital orthophoto plays great important role to take fast decision. Especially to control activities in large areas and countywide. Besides this, according to parallel development image processing technologies, it is essential to test an orthophoto for its accuracy (Bakıcı et al., 2008). The positional accuracy of spatial entities can be defined through measures of variations between the location of the feature registered in the database and the feature's exact location (Zuzelski et al., 2013).

2.2 Digital Orthophoto

Presently in the field of geomatics, digital orthophoto represents the shortest and fastest way to range the up to date digital base. These data can be very efficiently used in geodesy, urban planning, environment protection, public services, etc. In addition to this, it can be characterized by its accuracy, low price and above all means of information in the sense of photography**.** Besides this, large-scale orthophoto characterizes as a means of important source of information to control the spatial phenomenon in urban environment. Especially in the urban areas, it is about fast and relative cheap way to provide up-to-date spatial information than traditional cartographic techniques (Nedeljković et al., 2004). In different sources and quality of digital spatial data's are commonly integrated in GIS environments, by defining an undetermined level of global correctness in such schemes. Moreover, the advancement of geographic information system (GIS) in the mapping practice has formed a completely new type of user unlike from the traditional map user. In fact, it is not difficult to make decent orthophoto along small areas whereas attempting to produce of good and homogeneous orthophoto in large areas becomes a challenge. On the other hand, it is also problematic to scale the quality of digital orthophoto without standardized quality parameters. And, the geometric quality can be investigated by making an assessment with geodetic data from other sources and it is based on the quality of the used imageries interior and exterior orientation parameters, which are the maximum distance of used image parts from the nadir point and the horizontal and vertical accuracy of the used 3D model. Therefore, to determine the quality of the produced orthophoto mosaic obtained from the noisy digital surface model (DSM), an assessment of positional accuracy can be conducted through implementation of the Positional Accuracy Standards for Digital Geospatial Data (ASPRS). In the end, this standard was established by the ASPRS Map Accuracy Standards Working Group for a purpose of reviewing and updating digital map accuracy standards to conform to state of the art recent technologies (Silvester et al., 2019).

2.3 Accuracy Assessment

Accuracy can be well-defined through the degree to which information on a map or in digital database ties actual / true or accepted values. Therefore, the disagreement between the encoded and the actual value of a particular attribute for a given unit of object is taken as an error. In other words, it is matter concerning the quality of data and the amount of errors has in a data set or map. It is likely to consider the horizontal and vertical accuracy of the geographic position as well as to attribute, conceptual, and logical accuracy in a GIS database. Actually, the level of accuracy required for particular applications differs significantly. Which is extremely precise data can be very challenging and costly to produce and accumulate. Accurateness becomes continuously a relative measure, since it is constantly measured according to the specifications. To evaluate its appropriateness for use, one has to examine the data according to the specification and also consider the limitations of the specification itself. In fact, its deviation of an estimate from the true value can be quantified by the mean squared error which is the summation of the variance and squared bias for an estimate (Stanislawski et al., 1996). Furthermore, spatial accuracy is the exactness of the spatial element of the database. The metrics used depend on the dimensionality of the entities under consideration. For points, accurateness is defined with regard to the remoteness among the encoded location and actual location. Briefly, blunder can be defined in various dimensions: x, y, z and horizontal, vertical, total. Metrics of error are extensions of classical statistical measures such as mean error, RMSE or root mean squared error, inference tests, confidence limits, etc. But, for lines and areas, the condition gets more complex. This is because error is a combination of positional error (error in locating well-defined points along the line) and generalization error (error in the points selected to represent the line). The spatial position of an arbitrary entities defined within a GIS data layer has a locational error that spirits to termed by one of the primary parameters, positional accuracy (Caprioli et al., 2003).

2.4 Positional Accuracy

Positional accuracy characterizes the closeness of those values encoded in a geo-spatial record to the entity's real location in that system. The necessities of positional precision for a GDB are directly associated to its intended uses. In addition to this, it's also investigated by means of a statistical evaluation of random as well as systematic errors (DOD, 1990). And also, it can be quantified by means of the root mean-square error RMSE or by the mean value of errors and their standard deviation. For this reason, comparison with self-determining or independent source of better accuracy is the preferred process to make an assessment of positional accuracy (ANSI, 1998). The better quality of cartographic products has permanently been of great significance. It is together with logical uniformity, the quality component of geographic information most extensively used by the national mapping agencies (NMAs) and also the more commonly assessed quality element preference (Jacobson and Vauglin, 2002). Hence, it is a substance of renewed curiosity because of the capabilities accessible by the global positioning system GPS and having the necessity of a better spatial interoperability to support the spatial data infrastructures. In the end, dissimilar locational value behaviour of features of data sets means the presence of an interproduct positional alteration and an obstruction to interoperation (López and Gordo, 2008).

2.5 Horizontal Positional Accuracy Assessment

The horizontal correctness of a map can be distinct as the RMS error in terms of the project's planimetric survey coordinates (X, Y) for test points as determined at full ground scale of the map. Therefore, the root mean square error is the aggregate outcome of all errors including those has got through the processes of ground control surveys, map compilation and final extraction of ground magnitudes from the map. As a result, evaluating for horizontal accuracy compliance is conducted by comparing the planimetric easting and northing coordinates of well-defined ground points. This data becomes compared with the coordinates of the same points which have obtained by a horizontal check survey of better accuracy. Briefly, the term well-defined points concerns to features that can be sharply recognized as separate points. Conversely, points which are not well-defined (that is poorly-defined) are excluded from the intended accuracy investigation. In fact, poorly defined image points are features that do not have well-defined midpoints such as roads that intersect at narrow angles (U.S. National Map Accuracy Standards, 1941). In addition, Minnesota Department of Transportation, has Applied the NSSDA to evaluate the positional quality level of large scale data sets. The NSSDA needs to attain this; the horizontal and vertical RMSE is multiplied by 1.7308 and 1.96 factor respectively. The result becomes along in the horizontal and vertical accuracies of 0.181 and 0.134 meters respectively (PAH, 1999).

2.6 Photogrammetric Data Accuracy Assessment

Firstly, an aerial photogrammetry surveying taken from an airborne vehicle and it comprises of making accurate measurements from photos and further information bases to determine, in general, the relative locations of points. Consequently, this enables to find distances, angles, areas, volumes, elevations, and sizes and shapes of entities. Generating planimetric and topographic maps from photographs as well as the production of orthophoto from digital imagery becomes the most common applications of metric photogrammetry (Wolf et al., 2014). Besides, the consideration of airborne photography for certain mapping applications is subject to on the scale of photography. Customarily, the feasibility of positional and elevation accuracies using photogrammetric approaches and the scale of the ultimate map were determined based on mapping desires and practice. Therefore, if photogrammetric systems are to be used for parcel boundary mapping, the technical (accuracy) necessities of the final map should be considered. Nevertheless, this has to contingent on how to best abolish the geometric displacements and distortions (Siriba, 2009). In addition, high-level linear features, such as roads, rivers, and lakes in aerial photogrammetry, or line segments, circles and curves in architectural and industrial applications, are more suitable for subsequent processes since they comprises better information than feature points (Mikhail and Weerawong, 1997). Linear features geometric restraints which are very important in photogrammetric treatments have also to be considered. The use of linear features is mainly pertinent to mapping and updating of constructed areas, since man-made substances include lots of linear curves. Correspondingly, natural entities like waterside lines and shorelines as well as vegetation boarders offer a decent opportunity to apply feature based approach (Heikkinen, 2002). During change detection and map updating through images, the correspondence among vector map and image features have to be determined for exterior orientation. Regularly, ground control points and the equivalent image points are manually extracted from the vector map and the image. Accordingly, the image parameters can be determined by the space resection method. Though, certain ostensible points can be recognized in the map, which creates this solution less efficient in most cases. It is helpful if the linear features can be used as control information (Zhang et al., 2008). The factors for instance camera tilt, relief displacement, lens distortion, and atmospheric refraction consequences in scale difference of digital orthophoto which results horizontal blunder while trying to make record (Barrette et al., 2000).

2.7 Cadastral Mapping

The necessity of having reliable information in developing countries remains more significant even with inadequate resources. In addition, the absence appropriately organized land records in the real land transaction are another challenges being faced by various developing nations. The actual reason becomes indistinguishable delimitation of individual or group rights, insecure ownership. The necessity for a functioning land market opens the way not only for private development but also for public land acquisition and other means of ensuring that land is available for dwelling and other urban needs. For this reason, digital cadastral information system establishment and enactment in every national wide is an area which desires critical consideration. On the other hand, it is sensed that becomes a great necessity to at least initiate efforts towards achieving this formidable task in an organised manner. Because of preliminary exertions would pave the way to attain the desired objectives in the years to come and would yield us out of this status quo state. Further, to preserve the official records of regarding concerning to land parcels, their position, shape, size, land use, and ownership, the digital land cadastre becomes more crucial. As a result, these practices come out to a basis for property taxation that remains one of the main purposes of emerging digital land cadastre. Also, the information derived from digital cadastral maps can serve administration in several ways in addition to contributing to a comprehensive, equitable, and efficient land tax system (ALI and SHAKIR, 2012). Accordingly, the common scales for large scale mapping (cadastral maps) range from 1:500 to 1:10,000. The large scale maps usually for urban areas show more precise parcel dimensions and is often prepared by cadastral surveys for each parcel based on ground surveys or aerial photography.

2.8 GNSS for Accuracy Checking

The positional exactness of orthophoto talking about to the correctness at which the location coordinates (latitude, longitude and ellipsoidal height) of the spatial entities that are well standard on the orthophoto are expected in reference to their corresponding ground truth coordinates obtained at identical locations using an independent ground based in situ Global Navigation Satellite System (GNSS) observations (Congalton and Green, 2009).

Currently, the advancement of satellite centred Global Positioning System (GPS) has enabled to more practical, economic and accurate in 3-D positioning determination in geodesy and surveying works. Besides its accurateness, it has also the advantage of being applicable at local, regional and universal scales (Mohamed et al., 2010). Furthermore, GPS becomes an important instrument for mapping features because it makes to obtain accurate data at low cost relative to traditional surveying systems. The accuracy of digitized features is reliant on on the correctness of the mapping development with GPS (Heselton, 1998). To sample the spatial accuracy it is better to custom the Global Positioning System (GPS). Similarly, GPS also has a countless advantage to observe and compute accurate checkpoint location coordinate values. On the other hand, making evaluation of the computed location of the checkpoints with the location of their corresponding image on the digital orthophoto offers the desired spatial accuracy assessment (Greenfeld, 2001). For some GPS applications with high accuracy requirements, for instance geodynamics application and precise relative positioning over hundreds of kilometres, the exactness of the broadcast ephemerides is not adequate. Hence, ephemerides with a metre level accuracy are required. One effective solution is to use post processed ephemerides, which are usually generated with one week observations from a globally distributed tracking network (Shi, 1994). Nowadays, in numerous engineering applications areas the tradition of Global Positioning System (GPS) has been used effectively for surveying activities in multiple disciplines. Furthermore, web-based online services developed by numerous organizations; which are user friendly, unlimited and most of them are free; has become a substantial alternative approach against the high-cost scientific and commercial software on accomplishment of post processing and scrutinizing the GPS data (Ocalan et al., 2013). The average accuracies of the online services are found to be in the range of millimetres to decimetres accuracy subject on some aspects including the observation time. In fact, when observation has taken for longer sessions, the coordinate variation becomes decreases. The precision also varies based on observation type (single or dual frequency). Satellite availability also a significant factor to be considered. Although the practice of Global Navigation Satellite Systems (GNSS) has no significant role in strengthening the accuracy, in urban areas the number of satellite availability is very crucial component for internet based services (Adam, 2017).

2.9 Data Accuracy Standard

To examine digital spatial data, the American Society of Photogrammetry and Remote Sensing, ASPRS (2014) are the recommended standards to assess horizontal accurateness of an orthophoto for large scale map. Consequently, are the quality of orthophoto becomes ≤15 cm, 30 cm and ≥45 cm used for the purpose of Highest accuracy work use, Standard Mapping and GIS work use and Visualization and less accurate work use respectively.

According to Ethiopia context, the accuracy standard of the horizontal position of orthophoto established by the Ethiopia Geo-spatial Information Institute (EGII) is ±30 cm at the scale of 1:2,000; which is the endorsed scale for urban areas (Ministry of Urban Development and Housing, 2015). This agrees to two pixels at a Ground Sample Density (GSD) of 15 cm. On the other hand, the accuracy of the vertical position is ±45 cm, similarly corresponding to three pixels.

CHAPTER THREE

3 RESEARCH METHOLOGY

3.1 Description of the Study Area

Chiro Town is the administrative town of West Hararghe Zone. It is located in the Eastern Oromia and it accounts a total area of 6 km^2, 326 km far away from the capital city of Ethiopia, Addis Ababa. In terms of relative location, it is bounded by Alawagora Kebele in the East, Wedeyti kebele in the North, Nejabas Kebele in the West, and Chiro Kela in South. Geographically**,** it is located in the Amhar Mountains; it extends from 9.059 ^{0} N to 9.098 ^{0} N latitude and 40.856 ^{0} E to 40.876 ^{0} E longitudes and an altitude of 1826 meters above sea level. It is the administrative core of the West Hararghe Zone.

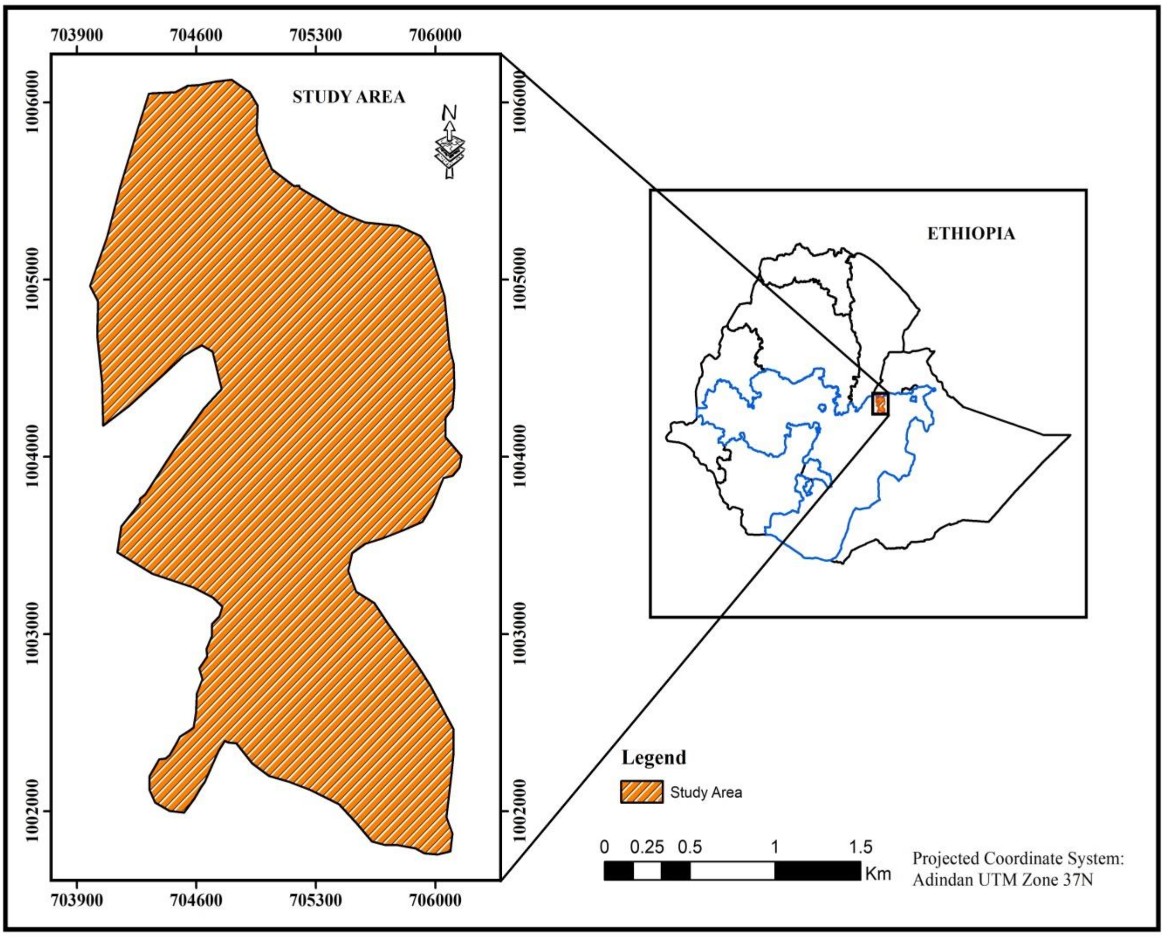

Figure 3.1: Location map of the study area.

3.1.1 Demographics

According to the 2007 national census report, a total population for this town of 33,670, of whom 18,118 were men and 15,552 were women. The majority of the residents said they were Muslim with 49.88% of the population reporting they perceived this credence; while 43.34% of the population practised Ethiopian Orthodox Christianity and 5.33% of the population were Protestant. In addition, the 1994 national census reported this town had a total population of 18,678 of whom 9,218 were males and 9,460 were females.

3.2 Data Source

This study used different types of data's to evaluate horizontal positional accuracy of an orthophoto. Of these, Digital orthophoto and sample GCPs are the data's which are used for this study. Accordingly, an orthophoto data obtained from aerial photogrammetric survey which has been done over the whole area of Chiro Town, accounting for a total area of 6 km^2. The photogrammetric surveying was operated by the Information Network Security Agency (INSA) and it is obtained from the Federal urban land and land related property registration and information agency (FULLPRIA). It has been taken on 04-Nov-16 at 6:35:12 PM using Microsoft Ultra-cam Eagle 100 Camera a pixel resolution of 15 cm with a scale of 1/2000. In addition, an orthophoto with a state of in local datum, which the reference ellipsoid is known as Clark 1880 (a= 6378249.145, 1/f =293.466307656). During the production of the spatial data set, EGII translational transformation parameters have been used. In addition, this study also used ground control points, which are principally important for geometric adjustment of the given high resolution aerial photo. Accordingly, observation of reference points has been taken with a static GNSS receiver (Leica GS14 dual frequency) for ten hours and post processed using offline using a stable reference (ADIS) GPS point of Ethiopia as well as online processing techniques. The accuracy of the measured GCPs is found to be in mm to sub centimetre accuracy.

3.3 Materials and Software's Used for the Study

Leica GS 14 Differential GPS, Leica Geo-office (LGO) 8.4, Global Mapper 11, Arc GIS 10.7 and Microsoft office (MS excel and MS word) were used to accomplish this study. The used materials as well as software's with their associated application areas are presented in the next table in more illustrative way.

Table 3.1: Materials and software's used with their applications.

Material	**Applications**
Leica GS 14 Differential GPS	For static GNSS observation
Software's	**Applications**
Leica geo office 8.4	For post processing of observed data
Global Mapper 11	For coordinate transformations
Arc GIS 10.7	To prepare map layout, for processing, analysis and visualization of spatial data
Microsoft office (MS excel and MS word)	To analyse, present and report the findings

3.4 Methodology

3.4.1 Check Point Sampling Design and Sample Size

In this case, in order to have an adequate representation of an assessment, a sample design with a robust statistical basis was necessary. This design should produce an unbiased estimate, being robust to different spatial patterns and densities.

Sample size and spatial distribution of checkpoints plays an important role in the accuracy evaluation of any geospatial data. A data set's accuracy is tested by comparing the coordinates of several points within the data set to the coordinates of the similar points from an independent data set of greater accuracy. For this purpose, the cluster sampling technique was used due to the fact that selection and distribution of the checkpoints (CP's) primarily depend on visibility and recoverability of well-defined points on an orthophoto by ground observation (Sisay et al., 2018). In addition to this the study also used international and national standards like ASPRS (2014), NSSDA and FGDC for designing the distribution of checkpoints and sample size. According to the FGDC and NSSDA (1999) the first conditions becomes points used for comparison purpose must be well-defined. In short, it has to be easy to find and measure in both the data set being tested and in the independent data set. The selected points will differ depending on the type of dataset and output scale of the dataset. For data derived from maps at scales larger than 1:5,000, plats or property maps, for example features like utility access covers, intersections of sidewalks, curbs or gutters make suitable test points. For survey data sets, survey monuments or other well-marked survey points provide excellent test points. Based on this the study used well defined points like ditches as well as side walk junctures. The second condition is twenty or more test points are required to conduct a statistically significant for better accuracy evaluation. In this study nine ground control points are used due to small coverage of the study area and land use change. Besides this, different scholars (Boye et al., 2016; Sisay et al., 2017) have used below twenty points to assess horizontal positional accuracy assessment. In addition to the above points, the independent data set should be three times more accurate than the expected accuracy of the test data set. Finally, this study tried to an ideal distribution of test point's allocation for at least 20 percent being located in each quadrant and test points has been spaced at intervals of at least 10 percent of the diagonal distance across the rectangular data set.

3.4.2 GPS Data Acquisition Procedures

3.4.2.1 Planning

Planning becomes the first step in any kind of Geodetic, topographic, cadastre and engineering application surveys. Planning and design of GPS surveys is required to ensure efficient execution of surveys in accordance to acceptable standards. Due to the reasons like accuracy required, receiver type, observing techniques, applications and processing software, it is almost crucial to establish good preparation for GPS surveys. In another words, it's also better to decide on when, where and what to do, types of instruments to be used, and what methods to be applied etc. Therefore, in this phase what is required is to plan and schedule a GPS project based on good and bad satellite coverage information. Besides, the planning has considered a number of factors. These include: project application, accuracy requirements, site access and restrictions, equipment resources, GPS procedure, network design and connections, data collection and adjustment techniques etc.

3.4.2.2 Reconnaissance

In this stage a field visit and inspection has been conducted where the investigator takes measurement in the area. It enabled to have an over view of the type of the area and to assess more correctly the type and magnitude of problems being encountered. Based on the standard course of action and after carefully discovered the area, nine well defined test points has been chosen as a reference ground control points. Furthermore, a site sketch and brief description has been prepared to identify the selected points in the site. In addition to this, site obstruction/visibility condition was considered which is the degree of difficulty in occupying points due to such factors as site access, multipath effects, and satellite visibility should has been anticipated. The selected check points have shown more illustrative way in the subsequent figure.

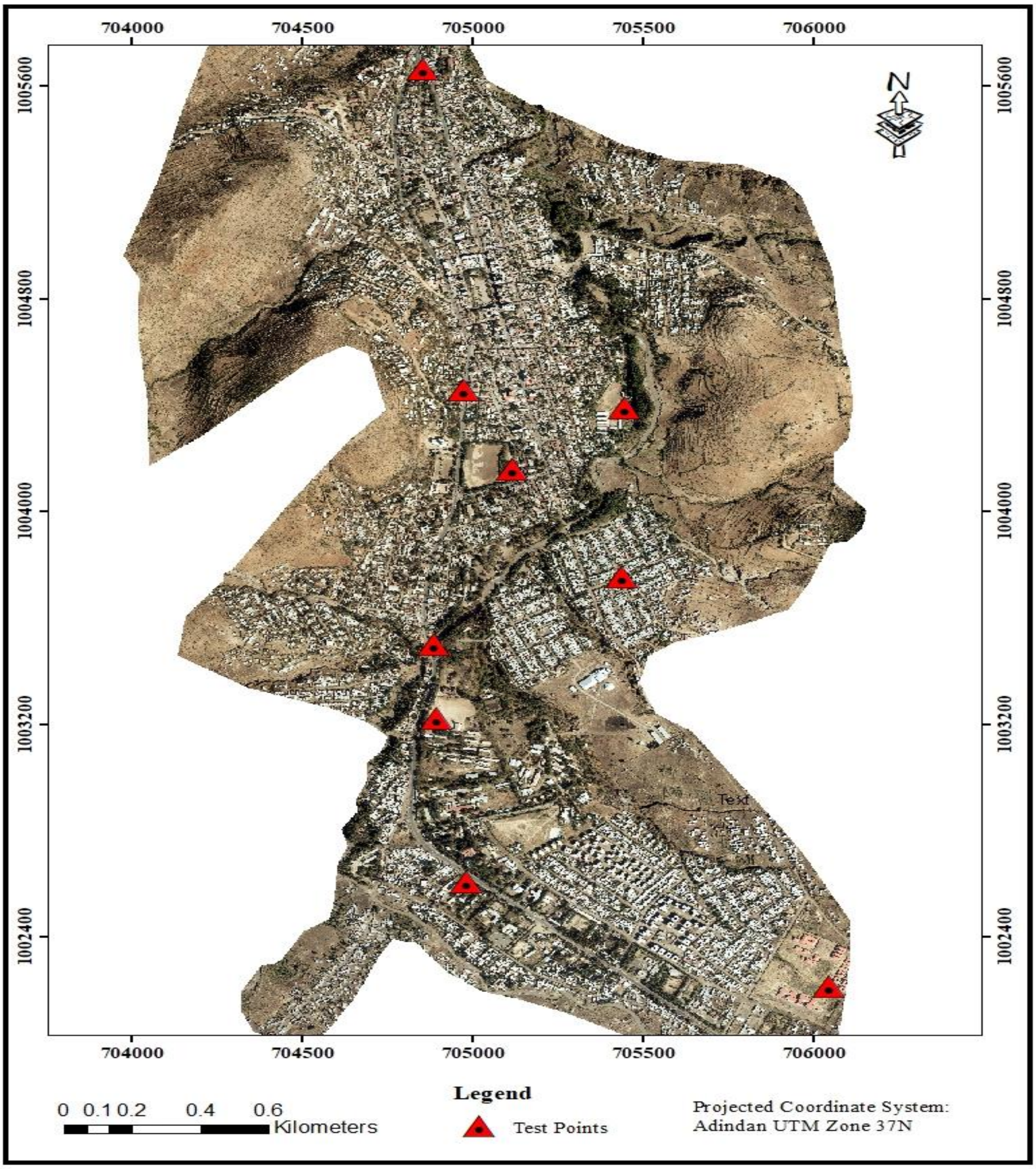

Figure 3.2: Checkpoints location.

3.4.2.3 GNSS Observation

In order to evaluate the horizontal accuracy of an aerial photo of the study area, observation of ground control points remained very crucial elements. For this reason, static GPS positioning technique was used. This technique delivers the highest accuracy attainable and necessitates the longest observation time. On the other hand, GNSS surveying with the static method is commonly used to evaluate high precision tri dimensional coordinates: these approaches offers coordinates of ground positions at a millimetre level both in the horizontal and vertical components (Correa-Muñoz and Cerón-Calderón, 2018). Besides, the ASPRS (2014) specified that the independent source of higher accuracy for checkpoints shall be at least three times more accurate than the intended exactness of the geospatial data set being verified. Accordingly, after carefully defined the test spots, the GNSS observation was taken using a Leica GS 14 GNSS receiver. The observation has been conducted for ten hours subject to the length of the base line, at a data logging interval of 10 seconds for each control points. ADIS CORS station (reference in Addis Ababa City) is far with a direct distance of around to 250 km with respect to the test points. Furthermore, the measurements were taken from six AM to five PM local time when there is no cloud that obstructs the satellite. During the measurements, a geodetic GNSS receiver with double frequency had a potential of concurrently tracking more than thirty satellites, both GPS and GLONASS were in the sky. Since, the locations of control points had clear moderately from obstructed horizons; no problems were encountered during the survey or measurement.

3.4.3 Post-Processing and Adjustment

It was also necessary to post-process the collected data using the GNSS data processing technique whether on line or offline for further processing. Firstly, the GPS observed DBX field data using DGPS instruments (Leica GS 14) were downloaded in an external memory and imported in to Leica Geo Office software. In addition to this, the reference RINEX data of the same day for each control points were downloaded. The reference, ADIS station, at the Geophysical Observatory of Addis Ababa University, was used. Besides, the reference receiver tracks both GPS and GLONASS satellites. The IGS GPS data and products are currently available, at no cost, to users worldwide through the Internet (https://cddis.nasa.gov/archive/gnss/data/daily/). The data is available in the standard RINEX format. In addition, to increase the accuracy of the ground control point's coordinates, the sp3 precise ephemeris data were downloaded from those stations via internet (https://cddis.nasa.gov/archive/gnss/products/) and imported into Leica geo-office 8.4 to facilitate the minimization of error using double differencing principle and least square sense. Since, field observation on the ground was conducted by Leica GS 14 instrument; it was well-suited to process using this software. Ultimately, the coordinates were processed and the basic control point's data were identified in WGS 1984 coordinate system.

Furthermore, this study also used GNSS online or Internet-based processing. This service is now extensively satisfactory as a substitute to traditional processing system (Adam, 2017). Internet based GNSS processing services namely: AUSPOS a free online GPS data processing facility provided by Geoscience Australia. Whereas, OPUS (Online Positioning User Service) of the US National Geodetic Survey (https://geodesy.noaa.gov/OPUS) and it uses baseline processing of standardized RINEX files into fixed positions. While AUSPOS, which is based on the Bernese software (https://www.gnss.ga.gov.au/auspos) and it allows the networked processing of observations from several user stations in RINEX format, using only remote IGS stations for reference. Accordingly, twelve IGS reference stations like ANKR, BHR4, DRAG, ISER, ISNA, MAL2, MBAR, NKLG, RAMO, REUN and TEHN have been used through this processing system. In fact, the AUSPOS processing facilities accurateness becomes consistent and better through the observation period when, the observations time longer the precision increase since the network of IGS stations which comprises of 12 stations is used for the processing (Tariq et al., 2017). On the other hand, in the circumstance of OPUS containing local reference four IGS reference stations has been used, these are ADIS, ISNA, BHR4 and SEYG. The reference stations were automatically chosen by both software. For this purpose, the GNSS observed data was converted from DBX file to the receiver independent exchange file format (RINEX 2.11). Then after, all converted points data has been uploaded on the AUSPOS and OPUS sites and processed results received via email. As a result both processing services were calculating the coordinates of each control points with a relative solution or double-differenced phase measurements method and deliver the ultimate coordinates with a precision of a few millimetres to a maximum error of sub centimetres.

3.4.4 Coordinate Transformation

In order to make an examination of horizontal accuracy assessment of orthophoto against check points first, it should have similar two-dimensional coordinate system type. Thus, Ethiopia uses local datum known as Adindan in which the reference ellipsoid is known as Clarke 1880 (a= 6378249.145, 1/f =293.466307656). Also, the datum transformation parameter provided by Ethiopia Geo-spatial Information Institute (EGII) the former Ethiopian Mapping Agency (EMA) was used to transform the GCP's coordinate from WGS 84 to Adindan and vice versa. The translation parameters 162 m, 12 m, -206 m have used been to transform global in to local coordinate system in X, Y and Z directions respectively. Conversely, to convert from Adindan to WGS 84 coordinate system, the translational parameters are -162 m, -12 m, and 206 m in X, Y and Z directions respectively (MUDH, 2015). Global mapper software was used to transform the processed GPS data from WGS84 coordinate system to Adindan coordinate system. Finally, the converted data has been saved in excel software as text files for data analysis purpose.

3.4.5 Methods of Data Analysis

Data analysis is the final procedure of the study. Accuracy assessment of digital orthophoto map is performed with absolute control of geometric accuracy, through comparison of the ground coordinates of reference points with the extracted control points on orthophoto map. The ground coordinates of reference points are determined from GNSS observation and are not included in digital aerial triangulation. In addition to this, using ArcGIS 10.7 the coordinates of the test points in orthophoto were measured on the most feasible location with respect to the ground observed spots. The measurements were repeated several times to avoid any misallocation and misinterpretation of the orthophoto test points. For the purpose of examining positional accuracy of orthophoto, all reference values obtained by using either the online or offline GNSS processing techniques were used. Therefore, According to ASPRS (2014) horizontal accuracy can be evaluated using root mean square error (RMSE) statistics in the horizontal plane, i.e., RMSEx, RMSEy and $RMSE_h$. In short, RMSE is the square root of the average of the set of squared differences among the dataset coordinate values and coordinate values from an independent source of higher accuracy for same points (Tsarovski, 2015). In other words, horizontal map accuracy is defined as the rms error in terms of the project's planimetric survey coordinates (X, Y) for test points as determined at full ground scale of the map. Consequently, the root mean square error is the aggregate outcome of all errors comprising those introduced by the procedures of ground control point surveys, map compilation and ultimate extraction of ground dimensions from the map. For examination purpose Microsoft excel software was used, all computations and pictorial descriptions were done using this software.

Therefore, using the 1989 NSSDA's standard the horizontal root mean square error (RMSEh) can be computed as follows:

The X and Y component positional RMSE is computed from:

$$RMSE_x = \sqrt{\sum_i^n ((X_{data\ i} - X_{check\ i})^2 / n)} \qquad \text{Eq. (1)}$$

$$RMSE_y = \sqrt{\sum_i^n ((Y_{data\ i} - Y_{check\ I})^2 / n)} \qquad \text{Eq. (2)}$$

Where:, $X_{data\ i}$ and $Y_{data\ i}$ are the orthophoto coordinates and $X_{check\ i}$ and $Y_{check\ i}$ are the reference GPS coordinates in the independent source of higher accuracy for the i^{th} sample point,,n is the number of check points tested, and i is an integer ranging from 1 to n.

Therefore, the horizontal root mean square error is defined as:

$$RMSE_h = \sqrt{\sum_i^n ((X\ data\ i - X\ check\ i)^2 + (Ydata\ i - Y\ check\ I)^2)} \qquad \text{Eq. (3)}$$

Or equivalently, the horizontal RMSE can be given as

$$RMSE_h = \sqrt{(RMSEx^2 + RMSEy^2)}$$

To compute horizontal accuracy at the 95% confidence level ($Accuracy_{h)}$, the accuracy value according to NSSDA, shall be computed by the following formula:

$$\text{Accuracy}_h = 1.7308 \cdot \text{RMSE}_h \qquad \text{Eq. (4)}$$

The horizontal accuracy statistic is determined by multiplying the RMSE by a value that represents the standard error of the mean at the 95 percent confidence level: 1.7308 when calculating horizontal accuracy, and 1.9600 when calculating vertical accuracy (PAH, 1999).

Thus, for this study the constant 1.7308 has been used to assess the horizontal positional accuracy of orthophoto, which are the standard value from the x-axis of the standard normal distribution for an interval with a probability of 95%.

The overall working procedure of the study has been shown on the following diagram.

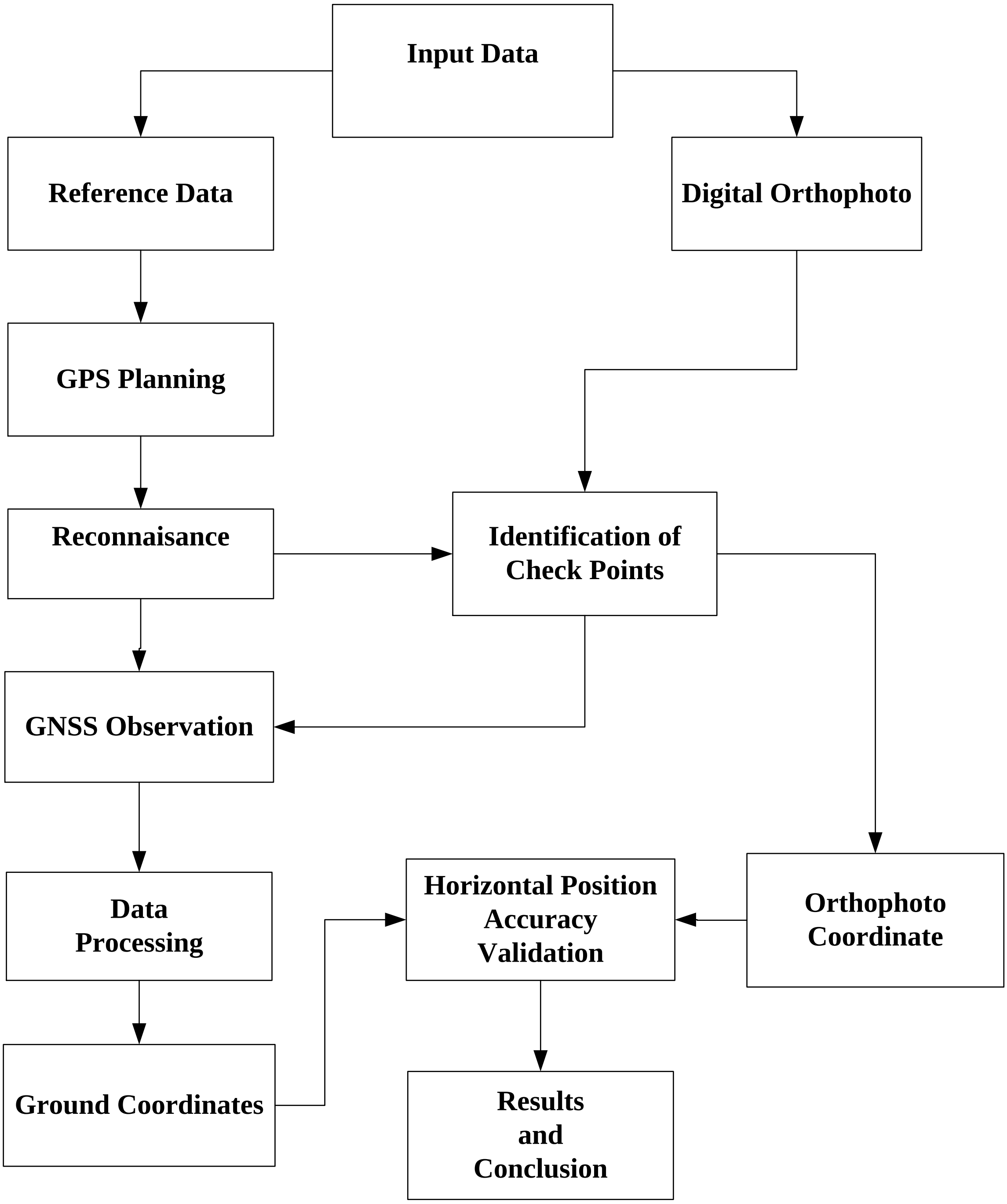

Figure 3.3: General work flow of the study

CHAPTER FOUR

4 RESULTS AND DISCUSSION

This study examined the accuracy of digital orthophoto in terms of horizontal position. Detail results of the accuracy assessment of orthophoto are discussed in the next sections.

4.1 Comparison of GNSS Processing Results

The observed GNSS data of nine check points was post processed using the two online services and Leica Geo office software and points accuracy found within millimetres to sub centimetre for all sessions and processing services. Accordingly, all GNSS data processing techniques (on line & offline) were used to evaluate self-internal accuracy in the GNSS data. As a result, the root mean square error (RMS) values of nine check points are 3.5 cm and 1.1 cm, 1 cm and 0.7 cm , and 5 cm and 1.9 cm along in x and y directions of LGO, AUSPOS and OPUS processing solutions respectively. Therefore, the results of the GNSS data processing indicates that AUSPOS result provide more accurate coordinates than others i.e. AUSPOS solutions has less root mean square error. The comparison values including mean error, standard deviations and root mean square error of all processing solutions has presented in the next table 4.1.

Table 4.1: Positional uncertainties of all GNSS processing results of nine check points.

Point	LGO		AUSPOS		OPUS	
	Easting (m)	Northing (m)	Easting (m)	Northing (m)	Easting (m)	Northing (m)
1	0.007	0.007	0.009	0.006	0.012	0.028
2	0.011	0.009	0.009	0.006	0.006	0.006
3	0.006	0.006	0.008	0.006	0.010	0.012
4	0.008	0.006	0.010	0.006	0.052	0.022
5	0.030	0.008	0.015	0.008	0.046	0.024
6	0.005	0.005	0.009	0.006	0.104	0.012
7	0.069	0.019	0.012	0.008	0.049	0.008
8	0.057	0.018	0.011	0.007	0.027	0.020
9	0.042	0.013	0.009	0.007	0.064	0.028
Min (m)	0.005	0.005	0.008	0.006	0.006	0.006
Max (m)	0.069	0.019	0.015	0.008	0.104	0.028
Mean (m)	0.026	0.010	0.010	0.007	0.041	0.018
Stdv. (m)	0.023	0.005	0.002	0.001	0.030	0.008
RMS (m)	0.035	0.011	0.010	0.007	0.051	0.019

The study also tried to show the comparisons among different processing techniques. Consequently, firstly LGO with the two internet based processing results has been assessed. Secondly, both AUSPOS and OPUS results has compared each other. While compared LGO with AUSPOS the standard deviations are 0.188 m and 0.019 m along in easing and northing directions respectively. The next three tables clearly show self-comparison results of all GNSS processing techniques along in easting and northing directions.

Table 4.2: Deviation in Easting and Northing of LGO and AUSPOS processing result of nine check points.

Point number	Easting (x)		diff in x	Northing (y)		diff in y
	(LGO)	(AUSPOS)		(LGO)	(AUSPOS)	
1	704971.606	704971.97	-0.364	1004458.676	1004458.910	-0.234
2	705442.658	705442.746	-0.088	1004394.512	1004394.779	-0.267
3	706042.821	706043.152	-0.331	1002215.970	1002216.221	-0.252
4	705434.652	705434.973	-0.322	1003757.852	1003758.092	-0.240
5	704882.639	704882.669	-0.030	1003503.604	1003503.881	-0.276
6	704892.534	704892.883	-0.349	1003227.273	1003227.548	-0.275
7	704853.432	704853.527	-0.095	1005665.677	1005665.935	-0.258
8	705113.474	705113.815	-0.341	1004164.236	1004164.473	-0.237
9	704977.254	704977.893	-0.638	1002610.788	1002611.009	-0.221
Sum (m)			-2.558			-2.260
Mean error (m)			-0.284			-0.251
Standard deviation (m)			0.188			0.019

In the same manner the results shows that when compared LGO with other on line processed results (OPUS) of which the standard deviations become 0.200 m and 0.019 m along in easing and northing directions respectively.

Table 4.3: Deviation in Easting and Northing of LGO and OPUS processing result of nine check points.

Point number	Easting (x)		diff in x	Northing (y)		diff in y
	(LGO)	(OPUS)		(LGO)	(OPUS)	
1	704971.606	704971.966	-0.360	1004458.676	1004458.907	-0.230
2	705442.658	705442.748	-0.090	1004394.512	1004394.780	-0.269
3	706042.821	706043.149	-0.327	1002215.970	1002216.220	-0.250
4	705434.652	705434.993	-0.341	1003757.852	1003758.086	-0.235
5	704882.639	704882.651	-0.012	1003503.604	1003503.880	-0.276
6	704892.534	704892.912	-0.378	1003227.273	1003227.540	-0.266
7	704853.432	704853.523	-0.091	1005665.677	1005665.922	-0.245
8	705113.474	705113.811	-0.338	1004164.236	1004164.468	-0.232
9	704977.254	704977.923	-0.669	1002610.788	1002611.013	-0.225
Sum (m)			-2.607			-2.227
Mean error (m)			-0.290			-0.247
Standard deviation (m)			0.200			0.019

For this study, two independent GPS data processing systems (on line & offline) were used to evaluate self-internal accuracy in the GPS data. The first solutions Table 4.2 and 4.3 shows the results determined by comparing LGO software with AUSPOS and OPUS processing results. While the second solutions are derived by AUSPOS using regional network of GPS reference stations taken over long baseline. And OPUS uses both local GPS reference stations and regional network of GPS reference stations taken over short and long baselines respectively. The results of the GPS data processing showed that both AUSPOS and OPUS has less standard deviation when comparing with each other. This indicates that results from both online processing facilities are very similar in practice. The deviation values are 0.017 m and 0.005 m along in easing and northing directions respectively. This is also shown in Table 4.4.

Table 4.4: Deviation in Easting and Northing of OPUS and AUSPOS processing result of nine check points.

Point number	Easting (x)		diff in x	Northing (y)		diff in y
	(OPUS)	(AUSPOS)		(OPUS)	(AUSPOS)	
1	704971.966	704971.970	-0.004	1004458.907	1004458.910	-0.003
2	705442.748	705442.746	0.002	1004394.780	1004394.779	0.001
3	706043.149	706043.152	-0.003	1002216.220	1002216.221	-0.002
4	705434.993	705434.973	0.019	1003758.086	1003758.092	-0.005
5	704882.651	704882.669	-0.018	1003503.880	1003503.881	-0.001
6	704892.912	704892.883	0.029	1003227.540	1003227.548	-0.008
7	704853.523	704853.527	-0.004	1005665.922	1005665.935	-0.013
8	705113.811	705113.815	-0.003	1004164.468	1004164.473	-0.005
9	704977.923	704977.893	0.031	1002611.013	1002611.009	0.004
Sum (m)			0.049			-0.032
Mean error (m)			0.005			-0.004
Standard deviation (m)			0.017			0.005

4.2 Horizontal Positional Accuracy Assessment

The horizontal positional accuracy of orthophoto was examined by comparing the two dimensional coordinates of the measured reference GPS points with their coordinates extracted in orthophoto. Statistical indices in this study can be used to evaluate, validate and compare the GPS data against digital orthophoto. The deviations of coordinates were computed applying the RMSE equations for eastings, northings and the overall horizontal positional accuracy. For the purpose of examining positional accuracy of orthophoto, all obtained values using the online as well as offline processing service are used as representative for all GNSS results. On the other hand, the ASPRS (2014) recommended that standards of horizontal accuracy for large scale map are ≤15 cm, 30 cm and ≥45 cm for highest accuracy work use, standard mapping and GIS work use and visualization and less accurate work use respectively. Therefore, this data set was tested to meet ASPRS Positional Accuracy Standards for Digital Geospatial Data (2014) for a Standard Mapping and GIS work use is 30 (cm) in RMSEx and RMSEy. However, the results show that the positional RMSE value of the nine check points in x and y-directions are 0.993 m and 1.462 m, 0.718 m and 1.712 m, 0.715 m and 1.709 m as compared to the orthophoto with GPS coordinate obtained from LGO, AUSPOS and OPUS processing respectively. And also which equates to 3.059 m, 3.213 m, and 3.206 m at 95% confidence level as compared to an orthophoto with the GPS data obtained from LGO, AUSPOS and OPUS processing respectively. The horizontal shifting of the extracted point coordinates from orthophoto as compared to that of the reference coordinates are presented in the next tables.

Table 4.5: Comparison of LGO GNSS data with that of Orthophoto.

Point number	Easting (x)		diff in x	Northing (y)		diff in y	(diff in x)^2+(diff in y)^2
	(LGO)	(ORTHO)		(LGO)	(ORTHO)		
1	704971.606	704972.550	-0.945	1004458.676	1004457.150	1.526	3.222
2	705442.658	705443.700	-1.042	1004394.512	1004393.100	1.412	3.078
3	706042.821	706043.858	-1.037	1002215.970	1002214.492	1.478	3.258
4	705434.652	705435.651	-0.999	1003757.852	1003756.349	1.502	3.256
5	704882.639	704883.300	-0.661	1003503.604	1003502.247	1.357	2.279
6	704892.534	704893.350	-0.816	1003227.273	1003225.650	1.623	3.301
7	704853.432	704854.499	-1.067	1005665.677	1005664.499	1.178	2.526
8	705113.474	705114.600	-1.126	1004164.236	1004162.849	1.387	3.194
9	704977.254	704978.402	-1.148	1002610.788	1002609.148	1.640	4.007
	Sum		-8.840			13.104	
	Min		-1.148			1.178	
	Max		-0.661			1.640	
	Mean error(m)		-0.982			1.456	
	Standard deviation (m)		0.156			0.143	
	RMSE (m)		0.993			1.462	
	$RMSE_h$ (m)						1.768
	NSSDA horizontal accuracy at 95% confidence level (m)						3.059

The results show that the root mean square error of easting and northing component is 0.993 m and 1.462 m respectively. These values correspond to horizontal $RMSE_h$ is 1.768 m. The root mean square error values in the easting, nothing and overall horizontal accuracy that are associated with the LGO GPS solutions with that of digital orthophoto are presented in more illustrative way using histogram.

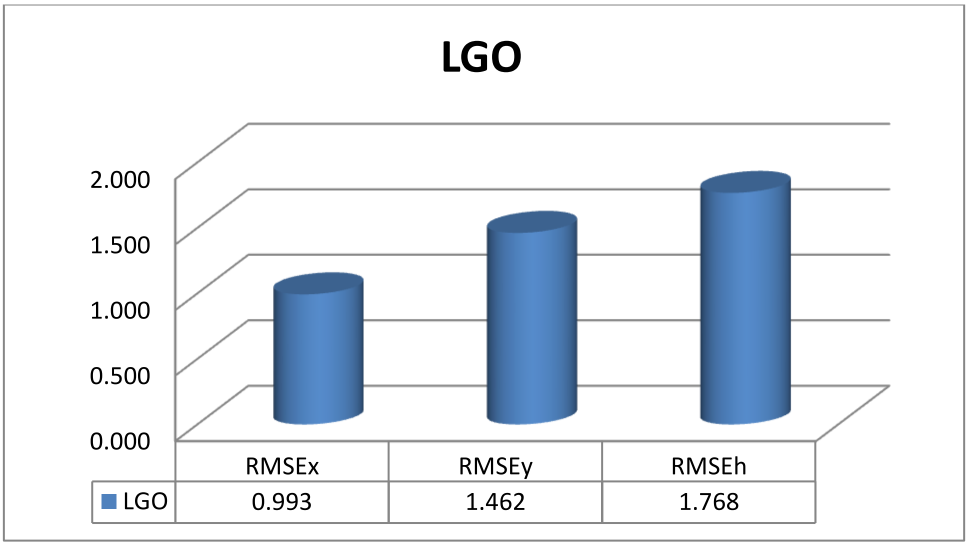

Figure 4.1: RMSE of the ground check points and digital orthophoto points using LGO processing

As discussed earlier, many factors will affect the spatial accuracy of an orthophoto among these factors are those that are related to image acquisition, control point selection and accuracy, and the rectification procedures. Because of this, the above and next table clearly indicates that the extent of horizontal shifting values between an orthophoto and in situ ground data. In this study, mainly focused in the end result or the impact of all the errors on the final accuracy of an orthophoto without investigating specific error sources. Therefore, a theoretical discussion on errors and their propagation into the orthophoto has not been presented here. Only the evaluation of the exiting errors in the Digital orthophoto has been discussed.

Table 4.6: Comparison of AUSPOS GNSS data with that of Orthophoto.

Point number	Easting (x)		diff in x	Northing (y)		diff in y	(diff in x)^2+(diff in y)^2
	(AUSPOS)	(ORTHO)		(AUSPOS)	(ORTHO)		
1	704971.970	704972.550	-0.580	1004458.910	1004457.150	1.760	3.435
2	705442.746	705443.700	-0.954	1004394.779	1004393.100	1.679	3.729
3	706043.152	706043.858	-0.706	1002216.221	1002214.492	1.729	3.489
4	705434.973	705435.651	-0.678	1003758.092	1003756.349	1.743	3.497
5	704882.669	704883.300	-0.631	1003503.881	1003502.247	1.634	3.067
6	704892.883	704893.350	-0.467	1003227.548	1003225.650	1.898	3.820
7	704853.527	704854.499	-0.972	1005665.935	1005664.499	1.436	3.005
8	705113.815	705114.600	-0.785	1004164.473	1004162.849	1.624	3.256
9	704977.893	704978.402	-0.509	1002611.009	1002609.148	1.861	3.723
	Sum		-6.283			15.364	
	Min		-0.972			1.436	
	Max		-0.467			1.898	
	Mean error (m)		-0.698			1.707	
	Standard deviation (m)		0.179			0.138	
	RMSE (m)		0.718			1.712	
	RMSE$_h$ (m)						1.857
	NSSDA horizontal accuracy at 95% confidence level (m)						3.213

The results show that the root mean square error of easting and northing component is 0.718 m and 1.712 m respectively. These values correspond to horizontal RMSE$_h$ is 1.857 m. The root mean square error values in the easting, nothing and overall horizontal accuracy that are associated with the AUSPOS GPS solutions with that of digital orthophoto are presented in more illustrative way using histogram.

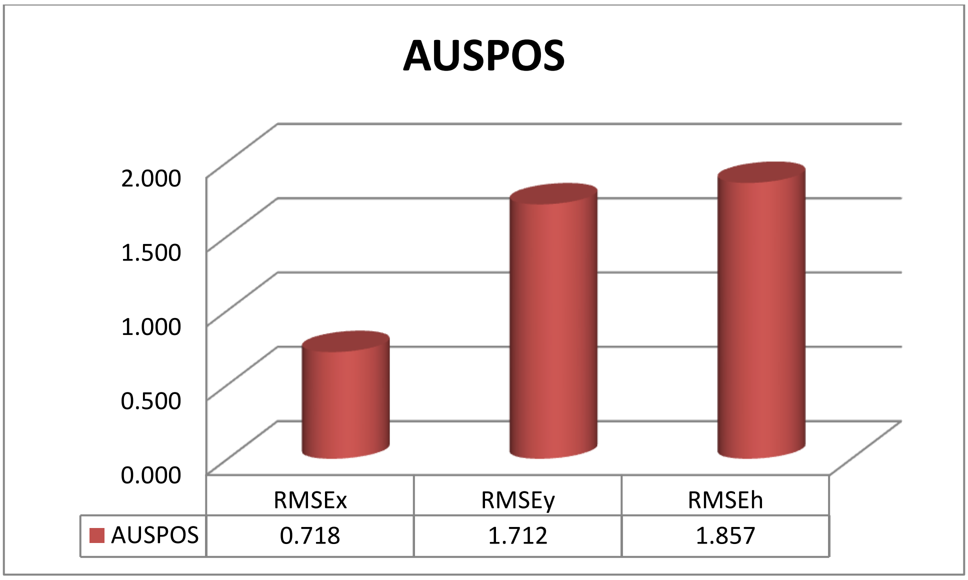

Figure 4.2: RMSE of the ground check points and digital orthophoto points using AUSPOS processing.

Table 4.7: Comparison of OPUS GNSS data with that of Orthophoto.

Point number	Easting (x)		diff in x	Northing (y)		diff in y	(diff in x)^2+(diff in y)^2
	(OPUS)	(ORTHO)		(OPUS)	(ORTHO)		
1	704971.966	704972.550	-0.584	1004458.907	1004457.150	1.757	3.427
2	705442.748	705443.700	-0.952	1004394.780	1004393.100	1.680	3.730
3	706043.149	706043.858	-0.709	1002216.220	1002214.492	1.728	3.488
4	705434.993	705435.651	-0.658	1003758.086	1003756.349	1.737	3.452
5	704882.651	704883.300	-0.649	1003503.880	1003502.247	1.633	3.088
6	704892.912	704893.350	-0.438	1003227.540	1003225.650	1.890	3.763
7	704853.523	704854.499	-0.976	1005665.922	1005664.499	1.423	2.976
8	705113.811	705114.600	-0.789	1004164.468	1004162.849	1.619	3.244
9	704977.923	704978.402	-0.479	1002611.013	1002609.148	1.865	3.707
Sum			-6.234			15.331	
Min			-0.976			1.423	
Max			-0.438			1.890	
Mean error(m)			-0.693			1.703	
Standard deviation (m)			0.188			0.140	
RMSE (m)			0.715			1.709	
$RMSE_h$ (m)							1.852
NSSDA horizontal accuracy at 95% confidence level (m)							3.206

The results show that the root mean square error of easting and northing component is 0.715 m and 1.709 m respectively. These values correspond to horizontal $RMSE_h$ is 1.852 m. The root mean square error values in the easting, nothing and overall horizontal accuracy that are associated with the OPUS GPS solutions with that of digital orthophoto are presented in more illustrative way using histogram.

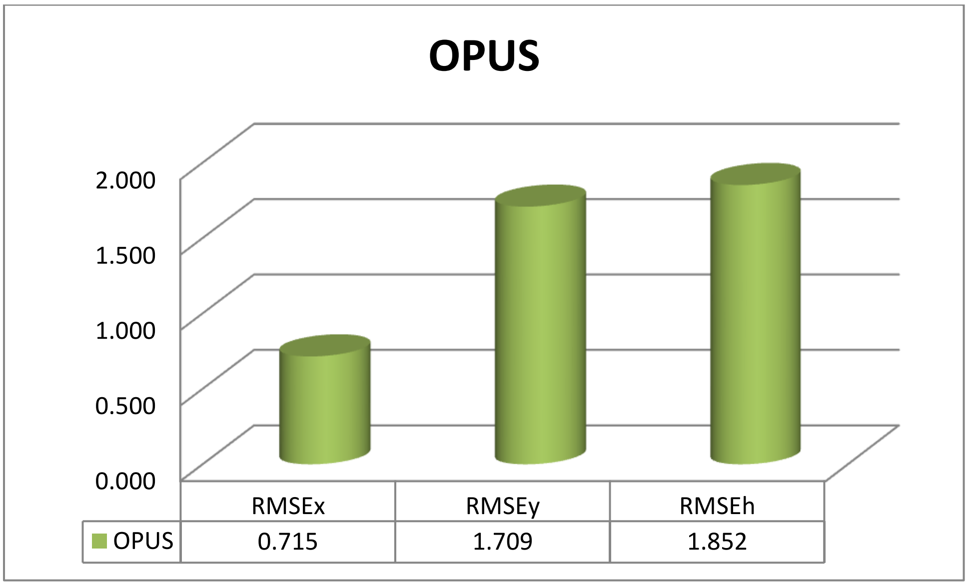

Figure 4.3: RMSE of the ground check points and digital orthophoto points using OPUS processing.

Furthermore, this study also analyses the root mean square error of easting and northing component as well as overall horizontal deviation of an orthophoto against all GNSS results. As we have seen on the above table 4.1 positional uncertainties of GNSS processing results value, AUSPOS processing solutions are more accurate value than LGO as well as OPUS solutions. Similarly, (Tariq et al., 2017) stated that the accuracy of AUSPOS processing services are regular because of the network of IGS stations which consists of 12 stations is used for the processing. In the same way, in this study the listed number of reference stations has been used by AUSPOS processing. On the other hand, the overall horizontal root mean square results show that while an orthophoto compared to LGO has less positional error than the internet based processing results. OPUS and AUSPOS are the subsequent results respectively. But, it's not to say that LGO processing solutions are more accurate value than the remained internet based processing solutions. It's simply when an orthophoto compared with obtained GNSS results of using the local

IGS (ADIS) as a reference has less positional error or gets more close value to the spatial dataset. Since, LGO processing using ADIS CORS as a reference. Whereas, AUSPOS using regional network of GPS reference stations taken over long baseline and OPUS uses both local GPS reference stations and regional network of GPS reference stations taken over short and long baselines respectively.

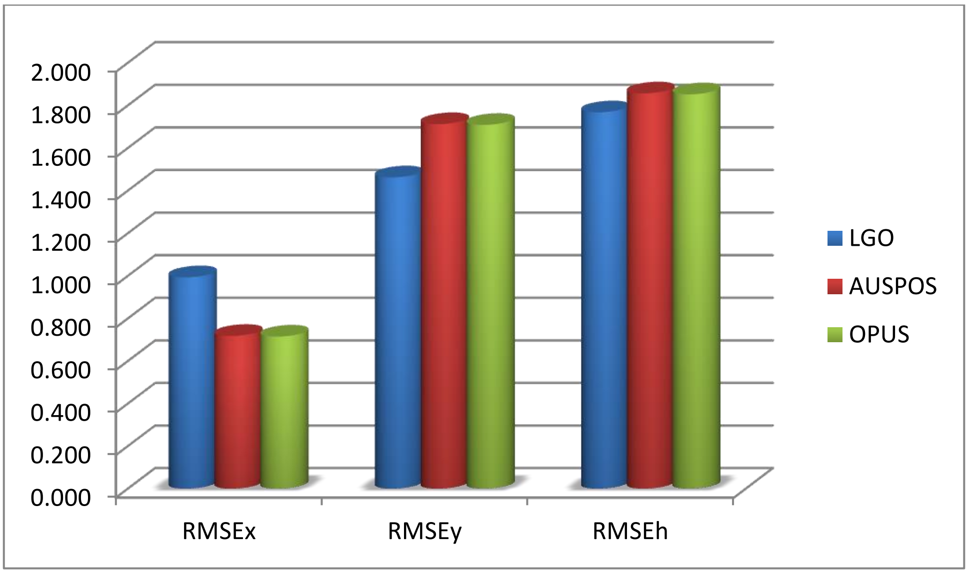

Figure 4.4: RMSE of the ground check points and digital orthophoto points using LGO, AUSPOS and OPUS processing.

Lastly, the NSSDA horizontal positional uncertainties at 95% confidence level result shows that the extent of positional errors of the dataset in x and y direction in all processing techniques. These errors, which emanate from the individual data, have a cumulative effect on the overall position accuracy determined. Therefore, the results of the CL at 95% value are 3.059 m, 3.213 m and 3.206 m as compared to an orthophoto with LGO, AUSPOS and OPUS processing respectively.

In Ethiopia some investigations regarding validations of an orthophoto has been conducted. Of these, (Sisay et al., 2017; Eniyew, 2018) evaluated the accuracy of orthophoto with in-situ GPS coordinates, results with a root mean square error (RMSE) of 12.45 cm in x and 13.97 cm in y directions and 18.1cm in x and 18.8 cm in y directions respectively. On the other hand, (Mulugeta, 2018) evaluated Dire Dawa City orthophoto in two approaches. Firstly, self-observed GNSS derived coordinates compared with orthophoto and it has a

RMSE of 0.67m and 2.498 m in x and y directions respectively. And also, he has tried to compare an orthophoto with existing GCPs in the town established by the Ethiopia Geo-spatial Information Institute (EGII) has s RMSE of 0.138m and 0.15m in x and y directions. Therefore, the first two studies (Sisay et al., 2017; Eniyew, 2018) has used CORS stations as a reference for their GNSS data processing and their evaluation with an orthophoto seems very close value and the accuracy of an orthophoto remains well for large scale mapping. On the other hand, according to (Mulugeta, 2018) we have seen that there is a large positional discrepancies between the exiting ground control points and new observed points. Similarly, in this study the accuracy of an orthophoto evaluated with a root mean square error of 0.746 in x and 1.685 in y directions using the most reliable solutions of AUSPOS processing services. However, the results of observed GNSS data to assess the given orthophoto were very precise solution. During offline processing ADIS CORS stations was used as a reference with an ambiguity has been fixed and well processed. In addition to this, during on line processing specially AUSPOS services has used twelve IGS stations as a reference and the final output coordinates has obtained within millimetres to a couple of centimetres. To sum up, based on the above study conducted in Dire Dawa City, this study undertakes using of the existing EGII ground control points at the time of orthophoto productions becomes the possible reason for the large horizontal position error for chiro town orthophoto. Because, as indicated by **(**Greenfeld, 2001) control points accuracy during orthophoto rectification process is one of the factors that affect its spatial accuracy.

Furthermore, in this study all results of RMSE value indicates that when compared an orthophoto with in situ ground measured results has a larger value in the y direction than in the x direction. In the same manner, this also clearly shown on the above examined outcomes on Dire Dawa city of new observed coordinates against an orthophoto. Therefore, the accuracy of ground control points coordinates makes an influence throughout orthophoto validation in each unit of components. However, only self-observed GNSS data has been used to assess the given orthophoto and it needs further study by incorporating existing EGII GCPs in the study area.

In the end, the spatial accuracy of the given orthophoto at a 95 percent confidence level was found to be 3.059 m, 3.213 m and 3.206 m compared with LGO, AUSPOS and OPUS processing solutions respectively. And also it’s presented in more illustrative way using histogram.

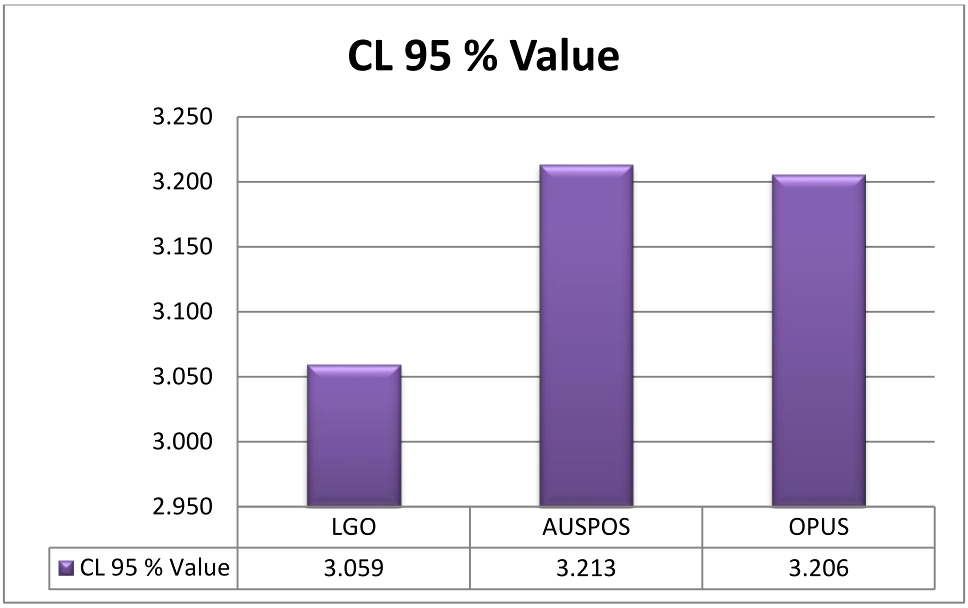

Figure 4.5: CL 95% values of the ground check points and digital aerial image points using LGO, AUSPOS and OPUS processing.

The question of the appropriateness of this orthophoto becomes more important. Using the validated orthophoto for large scale mapping like legal cadastral mapping has worthless unless making re orthorectification. As indicated by (Ministry of Urban Development and Housing, 2015), the accuracy requirement of horizontal position for large scale mapping becomes 30 cm or two pixels at a Ground Sample Density (GSD) of 15 cm pixel resolution. But also, in its present state, it can be used for a master plan and general economic development plan application. Because, those planning types requires only a general outline of where the parcels are located with respect to other features **(**Greenfeld, 2001).

CHAPTER FIVE

5 CONCLUSIONS AND RECOMMENDATIONS

The increased popularity of Digital orthophoto as a base maps for cadastral application necessitates an accuracy and quality assessment of that product. Consistently testing positional accuracy of orthophoto data using independent source of higher accuracy ensures the reliability of locational information which helps to validate decisions. Therefore, the purpose of this thesis work was to evaluate and compare the accuracy horizontal position of orthophoto. The comparison was made between GNSS points and an orthophoto. In this paper, we used a set of nine carefully selected check points was measured using static GNSS, processed using LGO software packages and two remote GNSS processing services. Finally, as we have seen on chapter four result sections, the horizontal RMSE and positional accuracy at 95% confidence level of the given spatial dataset was found 1.842 m and 3.189 m respectively between an orthophoto with the most reliable solution AUSPOS GNSS processing results.

5.1 Conclusions

Based on the results obtained, the following conclusions were made. For this study, LGO processing software and two remote GNSS processing services was used and their accuracy of each methods differ as compared each to them. As a result, based on the computation of positional uncertainties of each processing solution, AUSPOS services are the most reliable solution than the remaining processing results. In addition to this, as we have seen in the above result section, in all processing results the horizontal point location accuracies obtained doesn't meet the requirement of national as well as international standards for the purpose of Standard Mapping work use. Rather, it can use for Visualization and less accurate work use. Because, the accuracy which described in ASPRS (2014), and Ministry of Urban Development and Housing (2015), of ±0.30 m for a map scale of 1:2,000. But, in its present state, it can be used for a master plan and general economic development plan application. Furthermore, based on the results found the two online (internet based) processing results are close to each other and it indicates that both service solutions are similar in practice. Whereas, the offline processing results has less deviation with compared to an orthophoto but not to say that it is the reliable solution than others. LGO, OPUS and AUSPOS processing results are subsequently close to an orthophoto. This indicates that, while using local CORS (ADIS station) as a reference, it gets more close value to the aerial dataset. Since, during LGO, OPUS and AUSPOS processing, the extent of local reference used becomes completely, partially and nil respectively.

5.2 Recommendations

The obtained results from this study will hopefully improve the knowledge about evaluation of horizontal positional accuracy of orthophoto. Besides, the results also have shown which type of task can be conducted by using the analysed orthophoto. In addition to this, one can differentiate which GNSS processing technique are the most reliable slutions and has less deviations with compared to the original data set depending on the presented results. For further improvement of accuracy as well as further study, the following recommendations are forwarded. It is recommended that dependency on spatial dataset to support decision-making should be encouraged; however, such dataset should be subjected to rigorous positional assessment to enhance the reliability of such decisions. In addition, to increase the reliability of validation of orthophoto, it can be checked by using GNSS data which have used other different offline as well as internet based processing services solutions. Evaluating the existing established control points using new observed GNSS data and making an orthophoto validation using both coordinates becomes more important thing. The study also suggests that future study can be conducted using recent aerial photo to avoid current land use change problem in order to get better results. Finally, this study limited to evaluation of horizontal accuracy of orthophoto. But, can be further examined by incorporating the vertical component during line map extraction for 3D mapping applications.

REFERENCES

Adam, S.M. (2017). The Use of Online and Offline Processing Tools to Improve the Precision of a GPS Passive Station. Dept. of Engineering Surveying, College of Engineering ,University of Duhok, Kurdistan Region-Iraq, Journal of University of Duhok, Vol. 20, No.1 (Pure And Eng. Sciences), Pp 335-346.

ALI, Z. and SHAKIR, M. (2012). Implementing GIS-Based Cadastral and Land Information System in Pakistan. Journal of Settlements and Spatial Planning, vol. 3, no. 1, pp. 43-49.

American Society for Photogrammetry and Remote Sensing (ASPRS). (2014). Accuracy Standards for Digital Geospatial Data.

Bakıcı, S., Erkeka B. and Yıldırıma, Ö. (2008). An Investigation of Digital Orthophoto for Large Scale Mapping & Cadastre Renovation. The International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences. Vol. Xxxvii. Part B4.

Barrette, J., August, P. and Golet, F. (2000). Accuracy Assessment of Wetland Boundary Delineation Using Aerial Photography and Digital Orthophotography. Photogrammetric Engineering & Remote Sensing, Vol. 66, No. 4, Pp. 409-416.

Boye, C. B., Appeaning, K. and Williams, E. A. (2016). Positional Accuracy Assessment for Effective Shoreline Change Analyses. Ghana Mining Journal, Vol. 16, No. 1, pp. 1 - 7.

Caprioli, M., Scognamiglio, A., Strisciuglio, G. and Tarantino, E. (2003). Rules and Standards for Spatial Data Quality in GIS Environments. Polytechnic University of Bari, Via Orabona 4, 70125 Bari, Italy. Proceedings of the 21st International Cartographic Conference (ICC) Durban, South Africa, 10. 16 August 2003, .Cartographic Renaissance. Hosted By The International Cartographic Association (Ica), ISBN: 0-958-46093-0.

Chekole, S. D., Shibeshi, G. B. and Vries, W. T. (2020). An Evaluation Framework for Urban Cadastral System Policy in Ethiopia. Land 2020, 9, 60; doi: 10.3390/land9020060.

Correa-Muñoz, N.A. and Cerón-Calderón, L.A. (2018). Precision and accuracy of static GNSS for surveying networks used in Civil Engineering. *Ingeniería Investigation, 38(1)*, 52-59. DOI: 10.15446/ing.investig.v38n1.64543.

CSA (2007). *Population and Housing Census.* Oromia region, Ethiopia.

Drobnjak, S., Bankovic, R., Bakrac, S. and Kostic, M. (2017). Visualization of Horizontal Positional Accuracy Assessment Results for Digital Topographic Maps at Scale 1:25000. Doi: 10.15308/Sinteza-2017-308-314.

Eneyew, A. (2018). Horizontal Accuracy Assessment of Digital Aerial Photograph in Urban Cadaster: The Case of Wereda 10, Gulelie Sub-City, Addis Ababa. M.Sc. Thesis Addis Ababa University, Ethiopia.

FGDC, S. C. (1998). Geospatial positioning accuracy standards, part 3: National standard for spatial data accuracy.

Gavish, D. (1987). An Account of an Unrealized Aerial Cadastral Survey in Palestine under the British Mandate. The Geographic Journal, Vol. 153 (1), pp. 93 – 98.

Greenfeld, J. (2001). Evaluating the Accuracy of Digital Orthophoto Quadrangles (DOQ) in the Context of Parcel-Based GIS. Department of Civil and Environmental Engineering, New Jersey Institute of Technology, Newark, Nj 07102, Vol. 67, No. 2, Pp. 199-205.

Heselton, R.R. (1998). Elevation Effects on GPS Positional Accuracy. Faculty of the Virginia Polytechnic Institute and State University, Geography Department.

Hood, J., Ladner, L. and Champion, R. (1989). Image Processing Techniques for Digital Orthophotoquad Production. Photogrammetric Engineering and Remote Sensing Vol. 55, No.9, pp. 1323-1329.

https://geodesy.noaa.gov/OPUS/

https://www.gnss.ga.gov.au/auspos

https://www.bernese.unibe.ch/z

https://cddis.nasa.gov/archive/gnss/data/daily/

https://cddis.nasa.gov/archive/gnss/products/

López, F.J. A. and Gordo, A. D. A. (2008). Analysis of some positional accuracy assessment methodologies. Vol. 134, No. 2.

Ministry of Urban Development and Housing (MUDH). (2015), Urban Legal Cadastre Standard No. 03/2015.

Modiri, M., Enayati, H. and Ebrahimiki, M. (2015). The Assessment of Orthophoto Quality with Respect to the Structure of Digital Elevation Model. The International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences, Volume XL-1/W5.

Mohamed, E., Khaled, Z., Gamal, S. and Ibrahim, M. (2010). Accuracy Assessment of GPS Levelling Applications: Case Study, North West Lake Nasser, Aswan, Egypt. Vol. 2, Pp.147-160.

Mulugeta, E. (2018). Planimetric Accuracy Assessment of Digital Spatial Data for Large Scale Planimetric Mapping: The Case Study on Dire Dawa City, Ethiopia. M.Sc. Thesis Addis Ababa University, Ethiopia.

Ocalan, T., Erdogan, B. and Tunalioglu, N. (2013). Analysis of Web-Based Online Services for GPS Relative and Precise Point Positioning Techniques. Bol. Ciênc. Geod., sec. Artigos, Curitiba, V. 19, No. 2, p.191-207.

Nedeljković, I., Mihajlović, D. and Kukolj, D. (2004). Application of Large Scale Orthophoto in the Urban Changes Detection, MeCEO - First Mediterranean Conference on Earth Observation, April 21-23 2004, Belgrade.

Positional Accuracy Handbook (PAH). (1999). Using the National Standard for Spatial Data Accuracy to Measure and Report Geographic Data Quality.

Rufe, P.P. (2014). Digital Orthoimagery base specification V1.0: U.S. Geological Survey Techniques and Methods, Book 11, Chap. B5, 13 p.

Shi, J. (1994). High Accuracy Airborne Differential GPS Positioning Using A Multi-Receiver Configuration. Department of Geomatics Engineering, University of Calgary.

Silvester, S. Sai, Martinus, E. Tjahjadi and Catur A. Rokhmana (2019). Geometric Accuracy Assessments of Orthophoto Production from UAV Aerial Images. In the 1st International Conference on Geodesy, Geomatics, and Land Administration, Kne Engineering, Pages 333-344.

Siriba, D. (2009). Positional Accuracy Assessment of a Cadastral Dataset Based on the Knowledge of the Process Steps Used. 12th Agile International Conference on Geographic Information Science.

Sisay, Z.G., Bedada, T.B., and Gessesse, B. (2017). Feature Orientation And Positional Accuracy Assessment Of Digital Orthophoto And Line Map For Large Scale Mapping: The Case Study On Bahir Dar Town, Ethiopia. Volume XLII-1/W1, ISPRS Hannover Workshop: HRIGI 17 – CMRT 17 – ISA 17 – EuroCOW 17, 6–9 June 2017, Hannover, Germany.

Sisay, Z.G., Bedada, T.B., Gessesse, B. and Vermeer, M. (2018). Positional Accuracy Validation of Digital Orthophoto Mapping: Case Bahir Dar City, Ethiopia. Nordic

Journal of Surveying and Real Estate Research Volume 13, Number 1, 2018. https://doi.org/10.30672/njsr.65724.

Stanislawski, L.V., Dewltt, B.A. and Shrestha, R. L. (1996). Estimating Positional Accuracy of Data Layers within A GIS through Error Propagation. Surveying and Mapping, Department of Civil Engineering, University of Florida, 345 Weil Hall, Gainesville, Fl 32611-6580, Vol. 62, No. 4, April 1996, Pp. 429-433.

Tariq, M., Hadi, A. and Hafedh, H. (2017). Accuracy Assessment of Different GNSS Processing Software. Imperial Journal of Interdisciplinary Research (IJIR), Vol-3, Issue 10, 2017. ISSN: 2454-1362.

Weissmann, K. (1971). Photogrammetry Applied to Cadastral Survey in Switzerland. Photogrammetric Record, Vol. 7(37) 5 -15.

Wemegah, T.D. and Amissah, M.B. (2013). Accuracy Checks in the Production of Orthophotos. Journal of Environment and Earth Science, ISSN 2225-0948 (Online), Vol. 3, No.12.

Wolf, P.R., Dewitt, B. A. and Wilkinson, B. E. (2014). Elements of Photogrammetry with Applications in GIS. Fourth Edition, ISBN: 978-0-07-176111-6.

Zhang, Z., Zhang, Y., Zhang, J. and Zhang, H. (2008). Photogrammetric Modeling of Linear Features with Generalized Point Photogrammetry. Vol. 74, No. 9, Pp. 1119–1127.

Zuzelski, R. C., Agouris, P. and Doucette, P. (2013). A Photogrammetric Approach for Assessing Positional Accuracy of Openstreetmap Roads. ISPRS *Journal of Photogrammetry and Remote Sensing*, Int. J. Geo-Inf. 2013, 2, 276-301.

APPENDICES

Appendix 1: Row Data's Collected by Static GPS

6390_1219_042836.m00	19/12/2019 12:29	M00 File	1,265 KB
6390_1219_123229.m00	19/12/2019 14:50	M00 File	322 KB
7512_1211_034725.m00	11/12/2019 11:43	M00 File	1,276 KB
7512_1211_114404.m00	11/12/2019 15:21	M00 File	491 KB
7512_1212_043132.m00	12/12/2019 12:14	M00 File	1,239 KB
7512_1212_121532.m00	12/12/2019 14:55	M00 File	359 KB
7512_1218_042942.m00	18/12/2019 12:25	M00 File	1,285 KB
7512_1218_122602.m00	18/12/2019 15:02	M00 File	391 KB
7512_1219_034406.m00	19/12/2019 11:35	M00 File	1,217 KB
7512_1219_113612.m00	19/12/2019 15:02	M00 File	462 KB
7550_1210_050214.m00	10/12/2019 09:52	M00 File	534 KB
7550_1211_041959.m00	11/12/2019 11:53	M00 File	1,234 KB
7550_1211_115522.m00	11/12/2019 15:08	M00 File	448 KB
7550_1212_034843.m00	12/12/2019 11:38	M00 File	1,283 KB
7550_1212_114329.m00	12/12/2019 14:54	M00 File	448 KB
7550_1219_040509.m00	19/12/2019 11:10	M00 File	1,095 KB

Appendix 2: RINEX and Ephemeris Data

adis3440.19o	12/12/2019 10:21	19O File	5,090 KB
adis3450.19o	12/12/2019 10:22	19O File	4,711 KB
adis3460.19o	06/01/2020 13:13	19O File	5,056 KB
adis3520.19o	06/01/2020 13:22	19O File	5,077 KB
adis3530.19o	06/01/2020 13:05	19O File	4,692 KB
igs20832.sp3	14/01/2020 16:25	SP3 File	246 KB
igs20833.sp3	14/01/2020 16:25	SP3 File	247 KB
igs20834.sp3	14/01/2020 16:26	SP3 File	247 KB
igs20843.sp3	14/01/2020 16:28	SP3 File	248 KB
igs20844.sp3	14/01/2020 16:29	SP3 File	246 KB

Appendix 3: Opus Processing Solution Data

FILE: GPS13530.19o OP1612112566692

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: muhammedsiraj48@gmail.com DATE: January 31, 2021

RINEX FILE: gps1353e.19o TIME: 17:08:05 UTC

SOFTWARE: page5 2008.25 master95.pl 160321 START: 2019/12/19 04:29:00

EPHEMERIS: igs20844.eph [precise] STOP: 2019/12/19 14:50:00

NAV FILE: brdc3530.19n OBS USED: 16053 / 19946 : 80%

ANT NAME: LEIGS14 NONE # FIXED AMB: 146 / 190 : 77%

ARP HEIGHT: 1.27 OVERALL RMS: 0.023(m)

REF FRAME: ITRF2014 (EPOCH:2019.9655)

X: 4764583.040(m) 0.068(m)

Y: 4122421.116(m) 0.095(m)

Z: 1000322.308(m) 0.020(m)

LAT: 9 4 52.78498 0.020(m)

E LON: 40 52 1.46586 0.027(m)

W LON: 319 7 58.53414 0.027(m)

EL HGT: 1752.819(m) 0.113(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters]	1004371.586
Easting (X) [meters]	705206.233
Convergence [degrees]	0.29479444
Point Scale	1.00012113
Combined Factor	0.99984548

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ISNA			2547342.5
	BHR4			2146739.7
	SEYG			2216723.0

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13530.19o OP1612112244095

NGS OPUS SOLUTION REPORT

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

USER: muhammedsiraj48@gmail.com DATE: January 31, 2021
RINEX FILE: gps1353e.19o TIME: 17:02:54 UTC
SOFTWARE: page5 2008.25 master91.pl 160321 START: 2019/12/19 04:06:00
EPHEMERIS: igs20844.eph [precise] STOP: 2019/12/19 15:11:00
NAV FILE: brdc3530.19n OBS USED: 15046 / 20222 : 74%
ANT NAME: LEIGS14 NONE # FIXED AMB: 144 / 210 : 69%
ARP HEIGHT: 1.552 OVERALL RMS: 0.023(m)

REF FRAME: ITRF2014 (EPOCH:2019.9655)

X:	4764537.282(m)	0.020(m)
Y:	4122047.512(m)	0.062(m)
Z:	1001799.781(m)	0.014(m)

LAT:	9 5 41.69306	0.008(m)
E LON:	40 51 53.19616	0.049(m)
W LON:	319 8 6.80384	0.049(m)
EL HGT:	1710.644(m)	0.040(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters]	1005873.039
Easting (X) [meters]	704945.946
Convergence [degrees]	0.29486944
Point Scale	1.00011981
Combined Factor	0.99985079

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ISNA			2545907.3
	BHR4			2145530.0
	SEYG			2217898.8

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13530.19o OP1612112076825

NGS OPUS SOLUTION REPORT

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All computed coordinate accuracies are listed as peak-to-peak values.

For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: muhammedsiraj48@gmail.com DATE: January 31, 2021

RINEX FILE: gps1353d.19o TIME: 17:00:17 UTC

SOFTWARE: page5 2008.25 master94.pl 160321 START: 2019/12/19 03:45:00

EPHEMERIS: igs20844.eph [precise] STOP: 2019/12/19 15:01:00

NAV FILE: brdc3530.19n OBS USED: 18062 / 20905 : 86%

ANT NAME: LEIGS14 NONE # FIXED AMB: 122 / 162 : 75%

ARP HEIGHT: 1.432 OVERALL RMS: 0.022(m)

REF FRAME: ITRF2014 (EPOCH:2019.9655)

X: 4764895.842(m) 0.061(m)

Y: 4122501.479(m) 0.087(m)

Z: 998795.821(m) 0.030(m)

LAT: 9 4 2.25084 0.028(m)

E LON: 40 51 56.75522 0.064(m)

W LON: 319 8 3.24478 0.064(m)

EL HGT: 1797.584(m) 0.078(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters] 1002818.132

Easting (X) [meters] 705070.346

Convergence [degrees] 0.29413889

Point Scale 1.00012044

Combined Factor 0.99983775

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ISNA			2548877.9
	SEYG			2215802.7
	BHR4			2148172.8

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13520.19o OP1612111575462

NGS OPUS SOLUTION REPORT

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

USER: muhammedsiraj48@gmail.com DATE: January 31, 2021
RINEX FILE: gps1352e.19o TIME: 16:51:14 UTC

SOFTWARE: page5 2008.25 master93.pl 160321 START: 2019/12/18 04:31:00
EPHEMERIS: igs20843.eph [precise] STOP: 2019/12/18 15:02:00
NAV FILE: brdc3520.19n OBS USED: 20911 / 21872 : 96%
ANT NAME: LEIGS14 NONE # FIXED AMB: 80 / 99 : 81%
ARP HEIGHT: 1.367 OVERALL RMS: 0.018(m)

REF FRAME: ITRF2014 (EPOCH:2019.9627)

X: 4764859.985(m) 0.066(m)

Y: 4122362.215(m) 0.081(m)

Z: 999401.814(m) 0.012(m)

LAT: 9 4 22.32980 0.012(m)

E LON: 40 51 54.07546 0.104(m)

W LON: 319 8 5.92454 0.104(m)

EL HGT: 1776.357(m) 0.015(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters]	1003434.658
Easting (X) [meters]	704985.335
Convergence [degrees]	0.29420000
Point Scale	1.00012001
Combined Factor	0.99984066

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	SEYG			2216268.6
	BHR4			2147665.5
	ADIS			230838.1

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13460.19o OP1612110046741

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: muhammedsiraj48@gmail.com DATE: January 31, 2021

RINEX FILE: gps1346e.19o TIME: 16:26:07 UTC

SOFTWARE: page5 2008.25 master51.pl 160321 START: 2019/12/12 04:32:00

EPHEMERIS: igs20834.eph [precise] STOP: 2019/12/12 14:55:00

NAV FILE: brdc3460.19n OBS USED: 19706 / 22519 : 88%

ANT NAME: LEIGS14 NONE # FIXED AMB: 156 / 208 : 75%

ARP HEIGHT: 1.118 OVERALL RMS: 0.023(m)

REF FRAME: ITRF2014 (EPOCH:2019.9463)

X: 4764429.383(m) 0.078(m)

Y: 4122710.178(m) 0.031(m)

Z: 999920.694(m) 0.025(m)

LAT: 9 4 39.50558 0.022(m)

E LON: 40 52 11.91367 0.052(m)

W LON: 319 7 48.08633 0.052(m)

EL HGT: 1761.471(m) 0.063(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters] 1003965.204

Easting (X) [meters] 705527.415

Convergence [degrees] 0.29513611

Point Scale 1.00012277

Combined Factor 0.99984576

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	BHR4			2146954.7
	ADIS			231391.2
	SEYG			2216221.3

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13460.19o OP1612109906468

NGS OPUS SOLUTION REPORT

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

USER: muhammedsiraj48@gmail.com DATE: January 31, 2021
RINEX FILE: gps1346d.19o TIME: 16:24:50 UTC

SOFTWARE: page5 2008.25 master57.pl 160321 START: 2019/12/12 03:49:00
EPHEMERIS: igs20834.eph [precise] STOP: 2019/12/12 14:55:00
NAV FILE: brdc3460.19n OBS USED: 23203 / 23893 : 97%
ANT NAME: LEIGS14 NONE # FIXED AMB: 96 / 119 : 81%
ARP HEIGHT: 1.49 OVERALL RMS: 0.019(m)

REF FRAME: ITRF2014 (EPOCH:2019.9463)

X:	4764288.564(m)	0.040(m)
Y:	4123382.181(m)	0.031(m)
Z:	998409.101(m)	0.007(m)
LAT:	9 3 49.22399	0.012(m)
E LON:	40 52 31.56542	0.010(m)

W LON: 319 7 28.43458 0.010(m)

EL HGT: 1852.261(m) 0.049(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters]	1002423.338
Easting (X) [meters]	706135.570
Convergence [degrees]	0.29554722
Point Scale	1.00012586
Combined Factor	0.99983457

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE DISTANCE(m)
	BHR4		2148053.5
	ADIS		231971.1
	SEYG		2214773.9

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13450.19o OP1612109663992

NGS OPUS SOLUTION REPORT

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All computed coordinate accuracies are listed as peak-to-peak values.

For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: muhammedsiraj48@gmail.com DATE: January 31, 2021

RINEX FILE: gps1345e.19o TIME: 16:20:25 UTC

SOFTWARE: page5 2008.25 master92.pl 160321 START: 2019/12/11 04:22:00

EPHEMERIS: igs20833.eph [precise] STOP: 2019/12/11 15:09:00

NAV FILE: brdc3450.19n OBS USED: 20206 / 22344 : 90%

ANT NAME: LEIGS14 NONE # FIXED AMB: 112 / 140 : 80%

ARP HEIGHT: 1.253 OVERALL RMS: 0.023(m)

REF FRAME: ITRF2014 (EPOCH:2019.9436)

X:	4764322.026(m)	0.058(m)
Y:	4122631.877(m)	0.042(m)
Z:	1000544.354(m)	0.018(m)

LAT:	9 5 0.22531	0.006(m)
E LON:	40 52 12.27500	0.006(m)
W LON:	319 7 47.72500	0.006(m)
EL HGT:	1729.140(m)	0.073(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters]	1004601.898
Easting (X) [meters]	705535.170
Convergence [degrees]	0.29533611
Point Scale	1.00012280
Combined Factor	0.99985087

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	SEYG			2216631.2
	ISNA			2547073.1
	BHR4			2146386.3

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13450.19o OP1612109398117

NGS OPUS SOLUTION REPORT

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All computed coordinate accuracies are listed as peak-to-peak values.

For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: muhammedsiraj48@gmail.com DATE: January 31, 2021

RINEX FILE: gps1345d.19o TIME: 16:16:24 UTC

SOFTWARE: page5 2008.25 master54.pl 160321 START: 2019/12/11 03:48:00

EPHEMERIS: igs20833.eph [precise] STOP: 2019/12/11 15:21:00

NAV FILE: brdc3450.19n OBS USED: 19667 / 24210 : 81%

ANT NAME: LEIGS14 NONE # FIXED AMB: 132 / 184 : 72%

ARP HEIGHT: 1.323 OVERALL RMS: 0.023(m)

REF FRAME: ITRF2014 (EPOCH:2019.9435)

X: 4764636.717(m) 0.111(m)

Y: 4122281.967(m) 0.086(m)

Z: 1000613.201(m) 0.049(m)

LAT: 9 5 2.39118 0.028(m)

E LON: 40 51 56.87104 0.012(m)

W LON: 319 8 3.12896 0.012(m)

EL HGT: 1748.919(m) 0.143(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters] 1004666.025

Easting (X) [meters] 705064.388

Convergence [degrees] 0.29468056

Point Scale 1.00012041

Combined Factor 0.99984537

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE	DISTANCE(m)
	ISNA			2547073.2
	BHR4			2146543.7
	SEYG			2217020.5

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.

FILE: GPS13440.19o OP1612107453565

NGS OPUS SOLUTION REPORT

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All computed coordinate accuracies are listed as peak-to-peak values.

For additional information: https://www.ngs.noaa.gov/OPUS/about.jsp#accuracy

USER: muhammedsiraj48@gmail.com DATE: January 31, 2021

RINEX FILE: gps1344f.19o TIME: 15:42:04 UTC

SOFTWARE: page5 2008.25 master97.pl 160321 START: 2019/12/10 05:03:00

EPHEMERIS: igs20832.eph [precise] STOP: 2019/12/10 09:52:00

NAV FILE: brdc3440.19n OBS USED: 10088 / 10936 : 92%

ANT NAME: LEIGS14 NONE # FIXED AMB: 48 / 68 : 71%

ARP HEIGHT: 0.793 OVERALL RMS: 0.017(m)

REF FRAME: ITRF2014 (EPOCH:2019.9406)

X:	4764819.926(m)	0.067(m)
Y:	4122315.864(m)	0.050(m)
Z:	999672.069(m)	0.029(m)

LAT:	9 4 31.32494	0.024(m)
E LON:	40 51 53.78594	0.046(m)
W LON:	319 8 6.21406	0.046(m)
EL HGT:	1759.117(m)	0.077(m)

UTM COORDINATES

UTM (Zone 37)

Northing (Y) [meters]	1003710.998
Easting (X) [meters]	704975.074
Convergence [degrees]	0.29426667
Point Scale	1.00011996
Combined Factor	0.99984332

BASE STATIONS USED

PID	DESIGNATION	LATITUDE	LONGITUDE DISTANCE(m)
	BHR4	2147424.3	
	SEYG	2216455.8	
	ADIS	230833.4	

This position and the above vector components were computed without any knowledge by the National Geodetic Survey regarding the equipment or field operating procedures used.