Addis Ababa University

Addis Ababa Institute of Technology (AAiT)

School of Civil and Environmental Engineering

Post Graduate Studies in Geodesy and Geomatics

Accuracy Assessment of Geospatial Data for Cadastral Application: A Case of Addis Ababa City, Ethiopia

By:

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The undersigned have examined the thesis entitled 'Accuracy Assessment of Geospatial Data for Cadastral Application: A Case of Addis Ababa City, Ethiopia' presented by Melese Wondatir, a candidate for the degree of Master of Science in Geodesy and Geomatics and hereby certify that it is worthy of acceptance

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Declaration

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Post Graduate Studies in Geodesy and Geomatics

This is to certify that this research work entitled 'Accuracy Assessment of Geospatial Data for Cadastral Application: A Case of Addis Ababa City, Ethiopia' is my own work under the supervision of Dr. Getachew Tesfaye. This work has not been presented elsewhere for assessment. All relevant materials used in this research have been duly acknowledged.

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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.

Getachew Tesfaye (PhD)		
Advisor	Signature	Date

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List of Acronyms

1D: 1 Dimensional

2D: 2 Dimensional

3D: 3 Dimensional

AAIT Addis Ababa Institute of Technology

AAU Addis Ababa University

ANN: Artificial Neural Networks

ASPRS: American Society of Photogrammetry and Remote Sensing

ASTER Advanced Space born Thermal Emission and Reflectance Radiometry

BNP: Blue Nile Points

DEM: Digital Elevation Model

DGPS: Differential Global Positioning System

DTM: Digital Terrain Model

DMA: Defense Mapping Agency

ECEF Earth Center Earth Fixed

EGM08 Earth Gravity Model 2008

EPSG European Petroleum Survey Group

ECSU Ethiopian Civil Service University

FGDC Federal Geographic Data Committee

GCP: Ground Control Point

GSD: Ground Sample Distance

GII: Geospatial Information Institute

GNSS: Global Navigation Satellite System

GPS: Global Positioning System

IMU: Inertial Measurement Unit

ITRF International Terrestrial Reference Frame

LGO Leica Geo-Office

LIS: Land Information System

LSC: Least Squares Collocations

MRE: Numerical Models Multiplier Regression Equation

MCS: Minimum Curvature Surface

MSL: Mean Sea Level

NMAS National Map Accuracy Standards

NIMA: National Imagery and Mapping Agency

OAU Organization of African Unity

OHM: Orthometric Height Method

PDOP: Precise Dilution of Precision

UAV: unmanned air born vehicles

UNECA United Nations Economic Commission for Africa

USCGS United States Coast and Geodetic Survey

USDMA United States Defense Mapping Agency

WGS: World Geodetic System

RINEX: Receiver Independent Exchange Format

RMSE: Root Mean Square Error

SRTM: Shuttle Radar Topographic Mission

Abstract

Datum transformation is the transformation of coordinate points from one known coordinate reference system into another. The recent practice of geospatial data collection, management, and analysis in Ethiopia is in geocentric coordinates of a point defined based on a global datum (WGS84). However, Ethiopia uses Adindan as the recognized local datum. The current official transformation parameters in use by Ethiopia are slightly different from the parameters determined by previous research and adopted in widely used geospatial software packages. In addition to this, the rotation and scale changes are ignored in the transformation parameters. The current practice of cadaster in Addis Ababa city is below the accuracy level of all local and global geospatial data standard guidelines. The main purpose of this study is to assess and evaluate the Accuracy Assessment of Geospatial Data for Cadastral Application: A Case of Addis Ababa City, Ethiopia. Statistical analyses have been done for second-order *GCP, orthophoto, and cadastral parcel accuracy assessment analysis based on* ASPRS 2014 standards. The *positional accuracy of second-order ground control points, orthophoto, and cadastral parcels evaluated by checkpoints is determined as 0.47m, 1.326m, and 1.87m respectively at a 95 % confidence level. Finally, the software packages used to convert from global to local reference datum differ from one another, causing major changes in the values of specific points. Coordinates values of Orthophoto and static GCPs were converted from WGS84 to Adindan using (X = 162m, Y = 12m, and Z = -206m); the RMS error of easting and northing values were 0.32 m and 0.69 m respectively. On the other hand, the static GCPs coordinates were converted by using (X = 165 m, Y =11 m, and Z = -206 m) and compared with orthophoto coordinates, the RMS error of easting and northing values were 2.65 m and 0.78 m respectively. And, according to Molodensky-Badekas seven transformation parameters using (169.674 m in X, 14.801m in y, and -204.841 in Z), the RMS error of easting and northing values were 3.055m and 0.697 m respectively. Finally, the researcher recommends these parameters shall utilize as unique and constant values to improve data integrity for geospatial work such as cadastral application.*

Keywords: Adindan, Cadaster, Coordinate System, Ground Control Point, WGS84

CHAPTER ONE

1. Introduction

1.1. Background

A coordinate system can be defined as a set of numbers which identifies a position in space (Ayer & Fosu, 2008). A geodetic datum is the geometrical representation of the earth for uses of the coordinate reference system. Geodetic datum transformation is the process of transforming from one known coordinate reference system in to another (Dawod & Alnaggar, 2000). The shape of the actual real earth is very complex, and consists of landmass, seas, ice lands, oceans, etc. 67 % of the surface area of the earth is covered by oceans, therefore the shape of the earth are best fits to mean sea level (MSL) or the equipotential surface of the earth (geoid). And geoid is the height reference surface for vertical leveling, which is the equipotential surface of the earth surface. Due to the actual nature of the earth, the highest point of the earth exists in Mount Everest (8848 meters above mean sea level) (Malconian et al., 1990), and the deepest is around the Pacific Ocean (11,000 meters below mean sea level) (Stewart & Jamieson, 2019). Depending on this height variation of the topography with the highest and lowest points, it is difficult to represent the earth with in a simple mathematical model. The centrifugal force of the earth is pulling in outward directions, therefore the earth is bulged at the equator and flattened at the pole; geodesists represent the figure of the earth as a spheroid.

The rapid growth of utilizing Global Navigation Satellite Systems (GNSS) of global and 3D coordinates has necessitated the replacement of the old 2D horizontal geodetic control network for the east Africa countries. However, when setting up the geodetic framework in Ethiopia, only the horizontal position was determined, and the leveling of the geodetic network determined the orthometric elevation (Ephrem Belay et al., 2022). The horizontal geodetic control network of Ethiopia has been established by different methods such as triangulation and GPS. The geodetic control networks of the country are based on two main datum’s, namely, Adindan, and world geodetic system 1984 (WGS 1984) datums. The Adindan Clarke 1880 datum is locally recognized in Ethiopia with the semi-major axis (a) = 6378249.145 m and inverse flattening (1/f)

= 293.465 m (Archinal, 1992). The global datum of WGS1984 have semi-major axis (a) = 6,378,137m and inverse flattening (1/f) = 298.257 223 563 m. The horizontal and vertical geodetic control network was established from 1957 to 1961, and the area covers the Blue Nile river watershed. The survey has established a network with the first order, second-order, and third-order points by Ethiopian organization in combination with United States personnel's (USCGS, 1961). In this project, 365 triangulation stations (with ellipsoidal), and 905 permanent benchmarks of a level network are established (with Orthometric height).

The triangulation was established in Adindan Clarke 1880 spheroid. The vertical control network of Ethiopia originated from Alexandria, Egypt (Nyssen et al., 2016). And the level networks connect from Egypt to Sudan Ghedaref, Addis Zemen, Addis Ababa, and Asab. It has 1976 miles of distance coverage vertically referenced from mean sea level. In Ethiopia, the geodetic networks are classified into four different classes depending on the GPS reading time, zero-order, first-order, second-order, and third-order with the minimum reading time of 72 hours, 48 hours, 24 hours, and 12 hours respectively. According to the geospatial information institute report, currently in Ethiopia about, 30 zero-order, 150 first order, 1300 second-order, 5000 third order, and 4-course stations(but not functional) points are found.

Currently, the developments of geospatial data collection, management, and analysis need the availability of proper geodetic infrastructures for integrating different types and sources of coordinates without a shift in positions for multipurpose applications. Nowadays, in Ethiopia positions are mostly collected by GNSS data collecting instruments based on WGS84 reference systems. The data are then subjected to transformations and projections to a locally used system Adindan Clarke 1880 (Younis, 2020b). The conversions between different coordinate systems should be well defined to guarantee the consistency of the coordinates on all systems and tools, as well as it should be compatible for every datum.

Previous studies and pieces of literature (Tullu Besha, 2010) develop an absolute geo-potential height system for Ethiopia using airborne gravity data, the 2008 Earth Gravity Model (EGM08), and Shuttle Radar Topographic Mission (SRTM) digital elevation data in Remove-Compute-Restore techniques. And the seven transformation parameter between WGS84 and Clarke1880 in different height assumption methods was determined by (Abubeker Mohammed, 2019). Also, the evaluation of urban cadaster was studied in four selected pilot areas (Adama, Bahir Dar, Gondar, and Dessie cities). However, it faced major problems from the start and throughout its implementation process, the main reasons are, lack of legal framework, resources, technical experts, baseline maps, ground control points (GCP), awareness from higher officials, high turnover of skilled surveyors and GIS experts. However, until now there is no published literature on the area of datum transformation with the connection of cadaster.

Generally, in Ethiopia the geodetic infrastructure and the expected positional accuracy of the existing and implemented cadaster has low positional accuracy, it has no consistency and well-organized standards. Therefor this study aims to assess and evaluate the effects of inconsistency of transformation parameters on geospatial data (orthophoto) for cadastral application through different methods. The transformation parameter analysis results have been matched with different studies to ascertain good, consistence transformation parameter used for any geospatial data integration in selected study area.

1.2. Statements of the Problem

Various geospatial tasks, such as cadaster, photogrammetry, surveying, and geodesy, require coordinated transformation between distinct reference systems. Because topographic maps are created using the local datum Adindan Clarke 1880, and contemporary surveying data is obtained using the WGS1984 datum, this issue remains significant for geospatial scholars in Ethiopia. The Ethiopian Geospatial Information Institute (GII) is responsible for determining, modifying, and maintaining transformation parameters. The Ethiopian geospatial information institute supplies three translation parameters in eight (eight) collocated GCPs: X = 162m ±3m, Y = 12m ±3m and Z = -206m± 3m. Geographic software packages such as ESRI's ArcGIS use 165 m, 11 m, and -206 m in the X, Y, and Z directions, respectively, while EPSG (and QGIS) use 166 m, 15 m, and -204 m. Local (Abubeker Mohammed, 2019) .

In global positioning era, the accurate and absolute location of cadastral map is mandatory both in global (WGS84) and local datum’s (Adindan). Currently, coordinate positions are collected in using different instrument setting the reference system to WGS84. The collected data are then transformed and projected in to local reference system. Recent practice of cadastral mapping requires the acquisition of high resolution aerial photographs using either aircraft or UAV’s, ground control point collected by GNSS, and socio-economic data. The transformation between different coordinate systems should be well defined to guarantee the consistency of the coordinates on all systems and tools. The cadastral practice in Addis Ababa, Ethiopia, was started in 1992 G.C since then a number of attempts (e.g., in 2004, 2010, 2016, and recently in 2020) have been made to install modern cadastral system in the city. Still, only the 2004 and 2010 base map cadaster is used for land tenure applications.

However, the 2010 cadaster was originally produced from 20 cm resolution orthophoto. The operational uses of cadaster developed from such orthophoto have a positional shift, area variation, and dimension difference between the ground measurements. Based on the Addis Ababa land admiration office study, this shift occurs due to different reasons including, lack of sufficient number and distribution of GCPs during orthophoto production, the accuracy of the ground control points, and the ability to identify them on the imagery (interpretation), DTM editing error, the scale of aerial photography, inconsistency usage of datum transformation parameters (EIABCCD, 2015).

Currently, the positional accuracy and resolution of orthophoto are becoming best and effective. Now Geospatial Information Institute (GII) has 10cm resolution and 25cm resolution orthophoto. Comparing the overall RMS error of the current high resolution of the orthophoto with that of the old orthophoto; the current orthophoto have good positional accuracy, spatial resolution, and image quality. Due to this the current resolution of orthophoto and the 2010 base map cadaster did not overlap on certain reasons like datum transformation parameters inconsistency issue, accuracy of bench mark used, and the positional accuracy of orthophoto that used for cadastral application.

According to the Addis Ababa land administration office report, the positional shift between orthophoto and cadastral map has a large shift in Addis Ababa city; it reaches up to 2-meter shifts (EIABCCD, 2015). Since we are using global and local datums, there is a need for consistence transformation parameters for various fields of interests. That will be on acceptable positional error level of standards. Generally, the current cadastral practices of Addis Ababa city fall below geospatial data standards, lacking details on coordinate systems, datum, inconsistency of parameters, and directional information of parcel lines. Therefore consistent datum transformation parameters are required for any geospatial application. Therefore, this study aims to address the gap of consistence datum transformation parameters in the connection of geospatial software for cadastral application.

1.3. Objectives

1.3.1. General Objective

The general objective of this study is to assess and evaluate the Accuracy Assessment of Geospatial Data for Cadastral Application for Addis Ababa City.

1.3.2. Specific Objectives

The specific objectives of this study are

- To assess the horizontal accuracy of the orthophoto for Addis Ababa city
- To evaluate the accuracy of second-order ground control points for Addis Ababa city
- To evaluate and compare the positional accuracy of cadastral parcel corners with RTK survey control points

1.4. Research Questions

- How to evaluate the accuracy of orthophoto with static differential GPS reading?
- How to evaluate the accuracy of the current second-order ground control points?
- How to evaluate and assess the accuracy of cadastral parcel corners with RTK survey points?

1.5. Significance of the Study

Geospatial data become primarily used in various fields of disciplines, and it must be current, up-to-date, and reliable. Currently, within Ethiopia there are different geospatial software packages that utilized to convert coordinates from global to local. However, their transformation parameter varies from one system to other; therefore, it requires unique, reliable, and consistent parameters to customize at such software's. Also, accurate digital orthophoto and ground control points utilized for various applications such as; urban planning, land administration, rural and urban cadaster, and engineering applications.

Nowadays, the needs for cadastral information are essential for land tenure, land information system, legal land holding, transaction, taxation, etc. Due to this, the accuracy of the transformation parameter takes the grate role in the integration of the different geospatial data in different datum. The main role of this study is to reduce the positional shift of the cadaster caused by transformation of the coordinate from one datum to the other and to identify the gap between the cadastral base map and the current high-resolution orthophoto. This research can be utilized as a springboard for further studies for those are interested in the area. Above all, it has its contribution to solving the poor knowledge of spatial referencing in urban planning which is commonly called base map shifting.

1.6. Scope of the Study

In light of the current geospatial data application and practice, the inconsistency of transformation parameters are occurred in different geospatial application like, ground control point re-projection from global to local datum, road project work, and urban planning work. But, this study aims to evaluate the effects of inconsistency of transformation parameters on digital orthophoto and ground control points, specifically in the cadastral application, which has broader implications and advantages in this area.

1.7. Limitations of the Study

This study has a certain limitations like, Lack of raw aerial photograph images for Addis Ababa city in 2010 collected by HANSA, which enables to re-process the orthophoto by changing the GCP under aerial triangulation stage. Also, this study has the lack of check points, and duration of static GPS survey reading time on in situ measurement in order to meet the American Society of Remote Sensing and Photogrammetry standards (ASPRS). In addition, the coordinate system affects the quality of the analysis due to lack of having precisely defined national geoid model because; it had impacted the analysis in defining the height value of a point's coordinate in the local datum. Irrespective of the altercations committed, the study had found out a good result that had enabled to deduce consolidated conclusions of the analysis and state awakening recommendations to streamline the future of the parcel map positional quality.

1.8. Organization of Thesis

This thesis is organized into six chapters and a brief description of the chapters is presented below to get a summary of the general composition of the thesis. Chapter 1 is about Introduction, a general overview of the proposed topic and background, statements of the problem, objectives, research questions, scope, significances of the study, limitations are presented. And, Chapter 2 is about Literature Review: This section includes theoretical review, empirical review, and conceptual framework. In addition, Chapter 3 is focus on Research material and Methods: Description of the study area, data and software packages, method, and analysis. And, Chapter 4 – Results: This section includes the result of the present study. And, Chapter 5 is about discussion of Results parts, in this chapter discussion and justification based on the finding of the research output are presented in relation to previous studies. Finally, Chapter 6 is about Conclusion and Recommendations: Based on the result, this thesis concludes and recommends for feature works.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. An Overview of Geodetic Datum

As per Smith (1997) definition a geodetic datum is "the exact model of the earth: the size and shape of the earth and the origin and orientation of the coordinate systems used to map" (J. R. Smith, 1997). Traditionally, there are two types of datum are used: horizontal and vertical datum, based on ellipsoids and geoid respectively (Seeber, 2008). Horizontal Datum characterizes the relationship between the physical earth and horizontal coordinates such as latitude and longitude. The vertical datum may be a collection of particular points on the Earth with known heights either over or under mean ocean level. Particularly, to characterize a datum, the following will be required: An sign within the field (usually through a monument) of where the datum's beginning point (root) is found, the azimuth of a line interfacing the datum's beginning point to a secondary point, the exact definition of the model of the earth upon which the datum is based, and finally an amount known as the datum's geoid partition at the starting point. There are various sorts of transformation models and coordinate systems within the world. One commonly used datum to portray the whole earth is WGS 1984, which refers to the World Geodetic Framework of 1984. The best mathematical estimation of the shape and size of the Earth, on which coordinate systems are based on ellipsoids and geoids (Ayer & Fosu, 2008). In common, the impact of a transformation on a 2D or 3D object will shift from a simple alter of location and orientation (with no alter in shape or size to a uniform alter scale factor), to changes of the shape and size of distinctive degrees of nonlinearity (Mikhail, 1982).

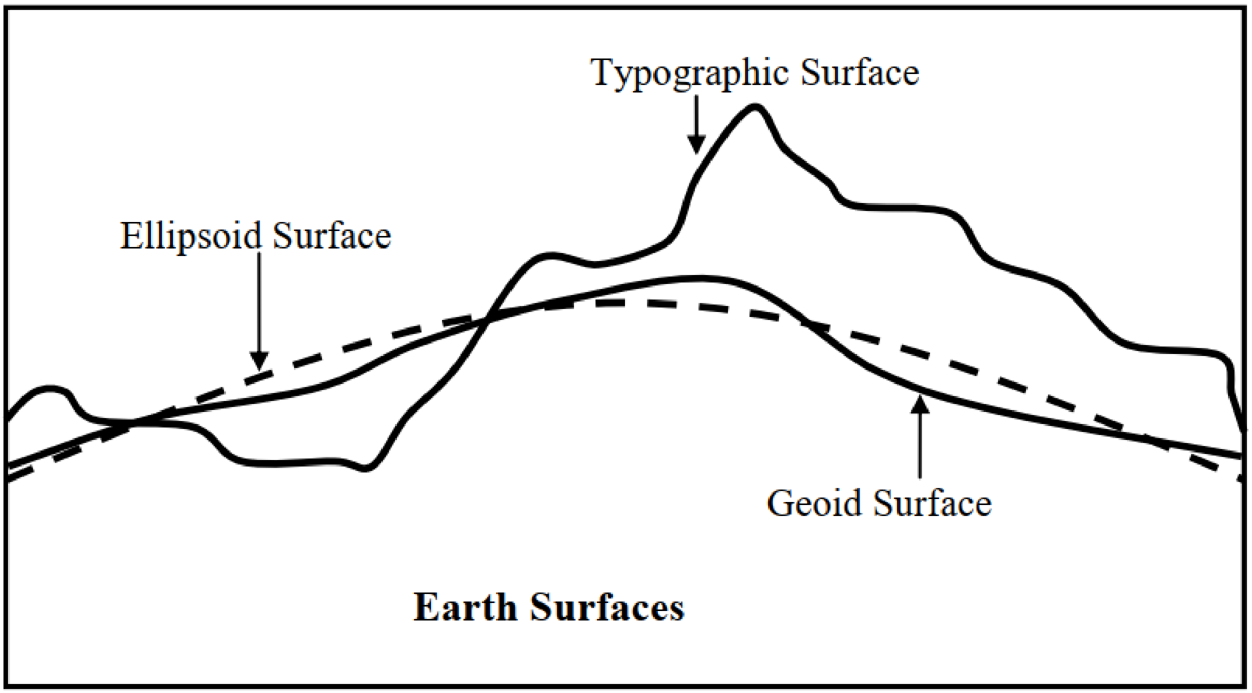

Figure 2-1 Earth Surfaces (Deng, 2015)

Generally, all these four points used a datum to establish a completely unambiguous set of references that can in turn be used to measure the location of any other point on the surface of the Earth.

2.1.1. Geoid

The geoid is the equipotential surface of the earth's gravity field that roughly approximates the mean sea level (Li & Götze, 2001). For height measurement above the earth surface a reference zero-height surface is required. This reference surface must be a level surface and for it to be used worldwide. The geoid has certain characteristics like, it is a physical definition, it can be defined by infinite number of parameters, it can be sensed by instruments, it is a complicated surface (irregular), it cannot be represented by any mathematical equation, and it enables users to determine the direction of water flow (Li & Götze, 2001).

2.1.2. Ellipsoid

Geoid is defined in infinite parameters, so it is difficult to mathematical model determination. Ellipsoids are the horizontal surfaces to which the geodetic latitudes and longitudes are referenced. Ellipsoid is that they fit the real shape of the earth as described by the geoid rather well and can thus be regarded as representative, yet simple, expression of the shape of the earth (Li & Götze, 2001). The ellipsoid has certain characteristics like it is, a mathematical definition, cannot be sensed by instruments, to be as close as possible (but not exactly) to the earth's surface (or geoid) on a national or global point of view, described by two parameters (the semi-major axis (for size) and the flattening (for shape)). Because the ellipsoid shape does not fit the Earth perfectly, there are lots of different ellipsoids in use. .

2.2. Coordinate Systems and Frames

To define a coordinate system we need to define: its origin (3 components), its orientation (3) components, and its scale (Dutton, 1999). The two coordinate types commonly used are ellipsoidal (geographic or curvilinear) coordinates (latitude, longitude and ellipsoidal height) and rectangular (Cartesian) coordinates - (XYZ).

2.2.1. Geographical Coordinates (ϕ, λ, h)

The most commonly used coordinate system today is the latitude, longitude, and height system whereas, the prime meridian and the equator are the reference planes used to define latitude and longitude. The position of a ground point can be defined by its geographic coordinates (ϕ, λ, h) (Shortis & Seager, 1994).

Where,

- Geodetic latitude ϕ : it is the angle between the equatorial plane and ellipsoidal normal;
- Geodetic longitude λ: it is the angle between meridian plane at Greenwich and meridian plane;
- Ellipsoidal height h: it is the height of the object above the reference ellipsoid along the ellipsoidal normal (Sonier & Chaumet, 1974).

2.2.2. Cartesian Coordinates (X, Y, Z)

It defines three- dimensional positions with respect to the center of mass of the reference ellipsoid. The X-axis is defined by the intersection of the plane defined by the prime meridian and the equatorial plane. The Y-axis completes a right- handed orthogonal system by a plane 90 degrees east of the X-axis and its intersection with the equator, and the Z-axis points toward the North Pole.

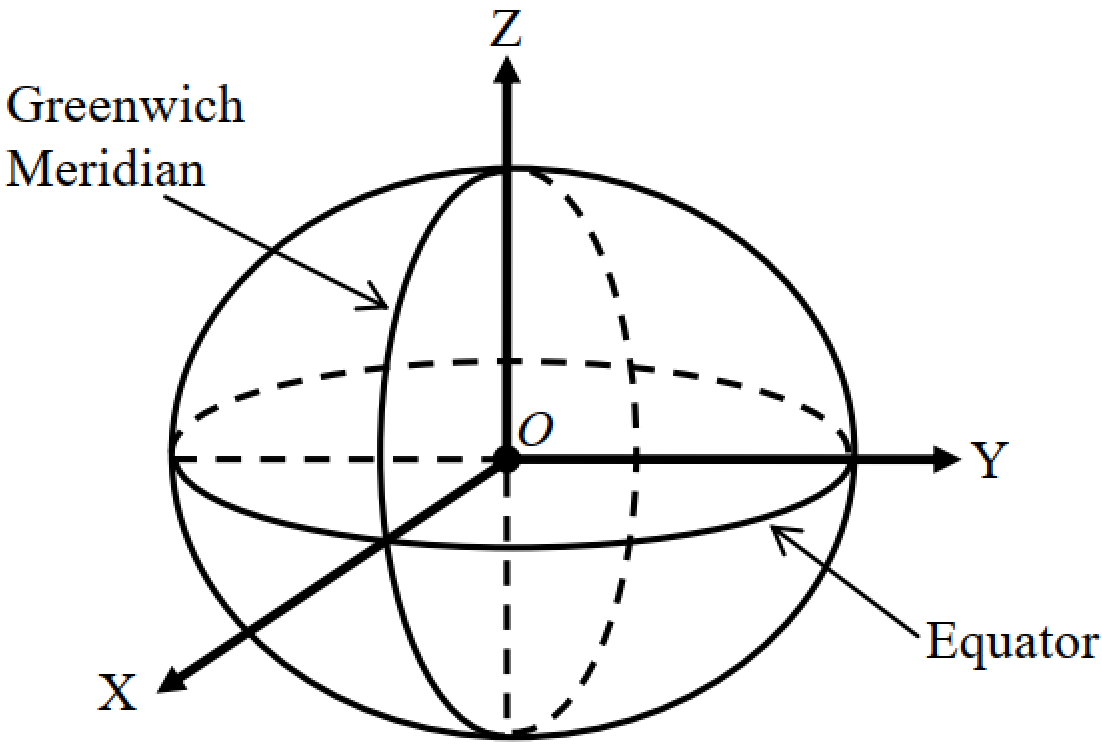

Figure 2-2 Earth Center Earth Fixed (ECEF) Coordinate System **: (**Deng**, 2015)**

Currently, there are many different coordinate systems in the world, based on different kinds of geodetic datum's units, projections, and reference systems that are in use today. The datum can be global like WGS-84 and local like Adindan-Ethiopia. Generally, Coordinate systems can be defined in three, five, seven, eight, or ten parameters. The relationship between the rectangular Cartesian coordinates (*x*, *y*, *and z*) and geodetic coordinates (ϕ, λ, h) can be expressed in Bowring's (1976) equations in equation (1) (Bowring, 1976) as:

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix} = \begin{vmatrix} (N+h)*cos\phi*cos\lambda \\ (N+h)*cos\phi*sin\lambda \\ (N(1-e\hat{}2\ )+h)*sin\phi \end{vmatrix} \tag{1}$$

Where 'e' denotes the first eccentricity of the reference ellipsoid and 'N' is the radius of curvature in the prime vertical at latitude ϕ.

Inverse computation from (ϕ, λ, and h) to (X, Y, Z) is complicated, for h is not in reference ellipsoid. Φ and λ and can be computed using closed formula of the Bowring's (1976) equations as(Bowring, 1976):

$$\tan\emptyset = \frac{Z + \frac{a.e^2}{\sqrt{1-e^2}}\sin^3\theta}{p - a.e^2.\cos^3\theta} \tag{2}$$

Where

$$\tan\Theta = \frac{z}{p\sqrt{1-e^2}} \tag{3}$$

$$\tan\lambda = \frac{y}{x} \tag{4}$$

2.3. Earlier Datum Transformation and Cadastral Approach

Numerous methods have been applied to determine datum transformation parameters and coordinate systems to deliver the best geospatial data integration. Anciently, ground measurements were utilized like, tape, level, and total stations to supply reliable data on the ground. In any case, it is very costly, time-consuming, labor-intensive, and usually troublesome to be connected to large-area assessments. Afterward, in later decades GPS has moreover been utilized to supply ground data. The differences of datums in utilize nowadays and the innovative progressions that have made conceivable global positioning estimations with sub-centimeter accuracies require cautious datum choice and cautious transformation between coordinate's completely different datums. The diversity of datums in use today and the technological advancements that have made possible global positioning measurements with sub-centimeter accuracies require careful datum selection and careful conversion between coordinates in different datums. Currently, Coordinate transformations are broadly utilized for various applications like cadaster, surveying, photogrammetry, and geodesy. For instance, three dimensional (3D) transformations used to transform from one reference system to another in geodesy (Andrei, 2006).

2.4. Digital Orthophoto

The aerial photograph is simply a photograph taken using aerial cameras mounted on the aerial vehicle like drone, aircraft, and helicopter (Gabrlik, 2015). An orthophoto is an aerial image that has been geometrically corrected. So It will be uniform throughout the image and has uniform scale (Linder, 2013). Presently in the field of geospatial science, digital orthophoto represents the shortest and fastest way to range the up to date digital base map cadaster. These data can be very efficiently used in geodesy, urban planning, environment protection, public services, etc. However, aerial photograph contains many distortions due to the lens, the camera's attitude, and the shape of the earth (Fusiello et al., 1997). All those distortions can be corrected through different software processing techniques to remove the effects of distortions that allow end-users to generate products that may appear as orthorectified images (orthophoto map).

Many factors affect the quality of digital orthophoto, these are quality of flight plan (which includes the mission plan, overlap, ground sample distance, flight altitude, weather condition, and other flight parameters), camera and bore sight calibration, reference station accuracy (which is like baseline distance and data logging time), the Inertial measurement unit (IMU), etc. (Tahar, 2013).

2.4.1. Flight plan

It is the first stage in aerial photography mapping processes, and the final product accuracy depends on the quality of an effective flight plan. According to Gibbons 20011 the required data for an effective flight plan is listed below (Gibbons, 2011).

- knowing the business purpose of the product

  It is a critical point to now the aim of the final product of the photo, the specification of the flight mission varies depending on the accuracy, resolution, GSD, and overlap parameters. For example, engineering application, urban planning, urban and rural cadaster, etc. needs different parameters and specifications.
- project boundary

  To analyze and decide the flight direction and flight strip project boundary is necessary; it is also used to see the coverage of the area by photograph, so it may be in shape file or .Kml file format.
- camera type

The calibration file, camera type, and focal length are also required for an effective mission plan.

- resolution(scale of photo)

  This is the key point to consider in an effective flight plan. The purpose of the product determines the ground sample distance (GSD) or the resolution.

- Overlap

  The forward overlap and the side lap planning are also required, which is the common area coverage between two successive photos in the same strip and adjacency strip respectively. The end lap is usually 60% plus or minus 5 %, but in most urban and rugged topography it may reach up to 80%. However the side lap is between 20%-40% plus or minus 10% for urban and rugged topography.

- DEM data

  To specify flight altitude the elevation data is required. The resolution and RMSE error of elevation data determine the vertical accuracy.

- Finally mission planning

  The number and distribution of ground control points (GCP), number of the strip, the total number of photos, final project cost, etc. determines the quality of the mission plan. for an effective mission plan, the main components are planning for precise dilution of precision (PDOP), reduced multipath, short baseline distance, static data collection, and inertial navigator alignment(Mostafa et al., 2001).

2.4.2. Calibration

The camera calibration report includes the interior orientation parameter, principal point offset, and focal length. Calibration can be either camera calibration to minimize distortion of the photo or boresight calibration which is used to measure the internal misalignment angle between the GPS and the camera center along x, y, and z-direction.

2.4.3. Aerial triangulation

Under this stage, the final radiometrically corrected and pan sharped individual photographs with their exterior orientation parameters are measured and rectified on external Pre marked or postmarked ground control points. Currently, within Ethiopia most of the GCP used to measure the orthophoto have 1 or 2 hour reading time. So the final good and well-distributed tie points are generated. Most of the research gaps were found under this stage because they did not consider the accuracy and the duration of logging time of GCP that is used for rectification of aerial photos. Because the final Ortho-mosaic photos RMSE errors depend on these GCPs.

2.4.4. Digital Terrain Modeling (DTM)

At this stage, 3D stereoscopic viewing of topographic modeling of bare earth terrain relief is edited. And it is removing vegetation, building, manmade and artificial features digitally. It will be done on either manually 3D mapping for better accuracy or fast automatic matching for limited accuracy comparatively.

2.4.5. Ortho-mosaic and Rectification

It is the final stage of the production of aerial photos. Geometric errors are removed that are caused by tilt and relief displacement. And it is an assembly of individual photos and stitching together into single mosaic images. Therefore based on the above photogrammetry stage the current 40% of the Ethiopia aerial photography coverage has been collected within 5cm-40cm Ground Sample Distance intervals. The above processing procedures and standards are currently utilized by Ethiopian Geospatial information institute (GII). Under this investigation, almost the entire archive photograph was collected under good RMSE, and it meets the international standards, which is Applanix standard.

Based on American Society for Photogrammetry and Remote Sensing (ASPRS), accuracy of aerial triangulation designed for digital planimetric data (orthoimagery and/or digital planimetric map) only(ASPRS, 2014):

- RMSEx (AT) or RMSEy(AT) = ½ * RMSEx(Map) or RMSEy(Map)
- RMSEz (AT) = RMSEx(Map) or RMSEy(Map) of orthoimagery whereas,

Accuracy of ground controls designed for planimetric data (orthoimagery and/or digital Planimetric map) production only:

- RMSEx or RMSEy(GCP) = ¼ * RMSEx(Map) or RMSEy(Map),
- RMSEz(GCP) = ½ * RMSEx(Map) or RMSEy(Map)

2.5. Orthophoto Positional Accuracy Assessment

Different works of literature define accuracy assessment in different ways but they have the same concepts. For example, the accuracy assessment is a key component of any project employing spatial data (Congalton, 2001). Accuracy assessment is the final step in the analysis of data which helps us to verify how accurate our result is and it determines the quality of the data (Jain et al., 2018). Positional accuracy is the closeness of coordinate's value to be assessed and the reference coordinate value that is assumed to be true or correct Humboldt State University (2019). *T*hese assessments can either be qualitative or quantitative. *F*or example qualitative is usually a quick comparison analysis in position shift between the aerially sensed data and corresponds to what is on the ground, whereas quantitative assessments attempt to identify and quantify error. We compare map data with a reference of ground truth data. Congalton & Green (2019) described that accuracy of spatial data assess for three reasons like, desire to know how good data you have made for the sake of satisfaction, and to compare various related data sets to test which is best. horizontal accuracy is the root mean square (RMS) error in terms of planimetric survey coordinates (X, Y) for checked points as decided at the complete ground scale of the map and vertical accuracy as RMS error in evaluation in terms of assessment datum for well-defined points only (Congalton & Green, 2019).

Positional accuracy is additionally classified into two these are horizontal and vertical accuracy. Even accuracy is the horizontal (radial) component of the positional accuracy of information set to a flat datum, at an indicated certainty level. To assess the horizontal and vertical accuracy of ortho-rectified image in situ ground control points will be collected with different processing software packages like GAMIT/GLOBK, AUSPOS, Leica Geo-Office, and APPS (Canavosio-Zuzelski et al., 2013).

2.6. GCP positional Accuracy Assessment

A geodetic reference framework is made up of fixed monument stations whose positions are precisely measured and mathematically characterized in relation to a common datum(NRC, 1983). Geodetic cadastral control surveys are usually performed to establish a basic control network from which cadastral mapping activities, photogrammetric mapping works and others supplemental surveying actions are to be derived (NRC, 1983). For cadastral surveying application, the framework provides an accurate and efficient means to describe the location of land parcels and their relationship to one another, and makes it possible to interpret, analyze, and disseminate compatible positional parcel information.

By compatible, it means related, or tied, to the geodetic reference framework. The relationship between characteristics in the framework is also known if the exact spatial relationship among the points in the framework is known. A good geodetic reference framework ensures that different types of data in a legal cadaster are linked with enough spatial accuracy. The accuracy of the control points used in orthophoto map processing must be sufficient to support all of the anticipated cadastral mapping, the most demanding of which will be those pertaining to the land parcels to be extracted from the orthophoto map, according to the procedure and standards for multipurpose cadaster (1983) (NRC, 1983).

2.7. Application of Cadaster

A Cadaster is normally parcel based and up-to-date land information system containing a record of interests in land (e.g. rights, restrictions and responsibilities) (Yomralioglu & McLaughlin, 2017). Cadasters can be divided into three groups based on the type, quality, and quantity of data they include. Fiscal, legal, and multipurpose cadasters are the three types of cadasters (Yomralioglu & McLaughlin, 2017). A fiscal cadaster is a database that contains information needed to collect property taxes, such as the location and value of a piece. A legal cadaster is a register that identifies the legal owner of each land parcel as well as the precise borders of each lot. A multipurpose cadaster is one that combines legal and fiscal cadaster data with information on land use, infrastructure, buildings, soil, and other elements into a single source.

In 1907, Ethiopia passed its first real estate registration proclamation. It established procedures for land registration, transfer, and issuing of title deeds, as well as property taxation and other matters (Zerihun Amdemariam & Zein, 2013). This proclamation was supplanted by immovable property registration articles in the 1960 Ethiopian Civil Code, which were marginalized by the 1975 urban land and extra housing reform, which altered the urban land tenure system. The City Government of Addis Ababa initiated a cadastral project in 1994 in response to the enactment of the 1994 urban land lease holding ordinance, with the goal of registering all property for revenue purposes. Some of the significant issues included the lack of clear legal frameworks, focal organizations, and the right use of technology. As a result, the city began implementing a modern real property registration and land (cadaster) system development project in early 2009.

In 2007, four selected pilot project regions over Ethiopian main cities: Dessie, Bahir Dar, Adama, and Mekele City, were subjected to modern cadaster technique. The lack of a legislative framework, resources, technical expertise, baseline maps, ground control points (GCP), awareness from higher officials, and a high turnover of qualified surveyors and GIS experts were all important issues from the start and throughout the implementation process (Asmamaw Yehun, 2017). The second re-new urban cadaster procedure in Ethiopia began in 2011 and is still ongoing. The present urban cadaster is being implemented in Addis Ababa City as a pilot project, with documentation and other requirements in place for all other cities. However, in large regions, there are delays in documentation, resource allocations, and excessive personnel turnover (Asmamaw Yehun, 2017).

2.8. Geospatial Data Accuracy Standards

To assess and validate the geospatial data there are known international standards that are currently used by scholars.

2.8.1. The US National Map Accuracy Standards

The change in mapping brought about by photogrammetry led to the formation of the US National Map Accuracy Standards (NMAS). From its beginnings in about 1920, photogrammetry for mapping grew in popularity in the 1930s, although it "did not receive wholehearted acceptance within the surveying and mapping profession for the next many years" (Marsden, 1960). In 1941, the United States established "United States National Map Accuracy Standards," which were identical to those issued by the American Society of Photogrammetry in 1939. Between 1943 and 1947, the standard was reviewed once more. There was only one class of accuracy requirement in its final form, and it was made plain that each agency was responsible for the conformance testing of its maps, with no obligation to comply. "For maps on publishing scales higher than 1:20,000, not more than 10% of the points tested shall be in error by more than 1/30 inch, measured on the publication scale; for maps on publication scales of 1:20,000 or smaller, 1/50 inch," according to the US National Map Accuracy Standards. These precision constraints apply only to the positions of well-defined points in all situations. Well-defined points are those that are easily visible or recoverable on the ground, such as monuments or markers, such as bench markings, property boundary monuments; a road, railroad, and other crossings; corners of large buildings or structures (or center points of small buildings); and so on (Marsden, 1960).

2.8.2. The ASPRS 1990 Accuracy Standards

The horizontal and vertical thresholds for large scale maps for X, Y, and Z ground-scale RMSE values are set by the American Society of Photogrammetry and Remote Sensing Groups. Again, these 1990 ASPRS standards apply RMSE limits for Class 1, Class 2, and Class 3 products to maps printed with the specified map size and contour spacing, so digital geo-space. Not suitable for data. (D. Smith & Heidemann, 2015). The ASPRS positional accuracy standard is set the number of the sample points per area of being tested, like the Areas of ≤ 500 will be 20 checkpoints, 501 – 750 will be 25 checkpoints, and 751 - 1,000 square kilometers are required to have 30 checkpoints(ASPRS, 2015).

2.8.3. The National Standard for Spatial Data Accuracy (NSSDA) Standards

It published by Federal Geographic Data Committee FGDC (1998). FGDC was developed to report accuracy of digital geospatial data at the 95% confidence level as a function of RMSE values in X, Y and Z at ground scale, unconstrained by map scale or contour interval(FGDC, 1998). Where four common FGDC standards are:

- The checkpoints that are used to validate are three times more accurate than those being tested.
- To test positional accuracy minimum of 20 sample points is required.
- The sample points will be put in clear and visible areas, and it will be well-defined points
- And the sample points must be in all quadrants and well distributed through the areas. 20% of the checkpoints are in each quadrant, and the distance between them will be more than *d* /10, where *d* is the diagonal dimension of the point on the map (Authority, 1998).

2.8.4. Ethiopian “Standards of Map Accuracy”

In addition to these international standards, the national standards that used to guide the positional accuracy validation in the ministry of urban development, housing, and construction of Ethiopia as well as Ethiopia mapping agency is, ±0.3m for horizontal and ±0.45m for vertical position for 1:2,000 scale of photographs. This means that the aerial photograph was taken under 15cm ground sample distance (GSD), resolution.

2.8.5. Local Experience

Except for an unpublished MSc thesis conducted in 2009, Determination of Parameters for Datum Transformation between WGS 84 and Adindan-Ethiopia, there are no published papers on methodological aspects of datum transformation in geospatial data for cadastral application performed across the Ethiopian region (Abubeker Mohammed.,2019). Using conventional and conformal transformation models, Abubeker Mohammed (2019) set out to determine datum transformation parameters between global (WGS84) and modified local (Clarke1880) reference ellipsoids for the Ethiopian region. It was conducted using five different conformal transformation models, with the results indicating that the Molodensky-Badekas seven transformation parameters with iteration solution is the most stable and appropriate transformation model for Ethiopia when compared to the other approaches.

In Ethiopia, the positional accuracy in varied topography was studied for three cities, Bahir Dar, Debre Markos, and Harer which was tested in 2019 with three different scenarios. The study was tested in varying control points with different scenarios, first scenarios with 10 control points, second scenarios with 15 control points, and third scenarios with 20 Ground control points within situ measurements for each city with a 12-hour duration of GPS logging time reading, and the data was least-square adjusted in Leica Geo Office(LGO). And the result shows that the RMSE values of all cities are beyond the national geospatial data standard limit(Zinabu Getahun et al., 2019).

Besides, in 2017 G.C the Bahir Dar city positional accuracy of digital orthophoto (15cm resolution) and digital line map was tested. To assess the horizontal positional accuracy of Bahir Dar city 5 checkpoints that are processed in two different processing software GAMIT/GLOBK and Leica GeoOfice (LGO), the result shows that GAMIT/GLOBK has a good root mean square error (RMSE) than that of LGO processing software. Similarly, the comparisons are taken between 32 coordinate values reading in RTK GPS measurement along the road center and digitalizing of the road center line on a digital map, so the deviation of two techniques results with the mean value meeting the national geospatial data standard limit of the country that is currently in use (Zinabu Getahun et al., 2017).

In addition, the vertical accuracy evaluation was studied in 2020 in Mekelle, Ethiopia, to evaluate photogrammetric digital elevation data and global elevation data. Photogrammetric digital terrain model (DEM), shuttle radar topographic mission (SRTM), TerraSAR-X twin satellite of TanDEM-X (TDX), Advanced Spaceborne Thermal Emission and Reflectance Radiometry (ASTER) against in situ measurements of ground control points (GCP) with orthometric height (that are computed from world geodetic system (WGS84)) have all been evaluated in this study. To validate the vertical accuracy of photogrammetric DEM and global DEM, 2130 sample points were taken. As a result of this research, the photogrammetry DEM, when compared to the ground sample, had lower RMSE values than the others(Hareya Biryhane, Tullu Besha, Birhan Gessesse, & Vermeer, 2020) .

CHAPTER THREE

3. Materials and Methods

3.1. Description of the Study Area

3.1.1. Location

Ethiopia's capital and largest city, Addis Ababa, is situated in the country's central region. Addis Ababa is located at 8° 49' 48" up to 9° 6' 20" north latitude and 38° 38' 24" up to 38° 54' 23" east longitude as shown in (Figure 3-1). Ethiopia's capital city, Addis Ababa, is almost in the center of the country. As indicated in (Figure 3-1), the city has a total size of 539 km^2 and is divided into 11 sub-cities. Many continental and international organizations have their headquarters in Addis Ababa. It hosted continental organizations such as the Organization of African Unity (OAU) and international organizations such as the United Nations Economic Commission for Africa (ECA), earning it the moniker "Africa's diplomatic capital." The Addis Ababa city selected for this evaluation study due to its rugged topography, the highest population growth which needs to an effective utilization of land management, land banking and cadaster application. Besides, the Addis Ababa city orthophoto has a number of problems like; the radiometric correction error (has many dead pixels); which is difficult to digitize the parcel corners, distortion, and positional shift error. The aerial shot of Addis Ababa city was taken by HANSA in 2010. There were restricted on the 2010 E.C area of the city during the accusation of the Arial photo coverage at the time. Due to the city's urban expansion, regions not covered by the orthophoto are now collected using a total station and GPS, resulting in inconsistency and a positional shift for cadastral parcel formation. To maintain accuracy, uniformity, and acceptance for geospatial researchers, the effects of these established characteristics are only examined at the city level in Addis Ababa. The study area is graphically showed clearly on the next (Figure 3-1).

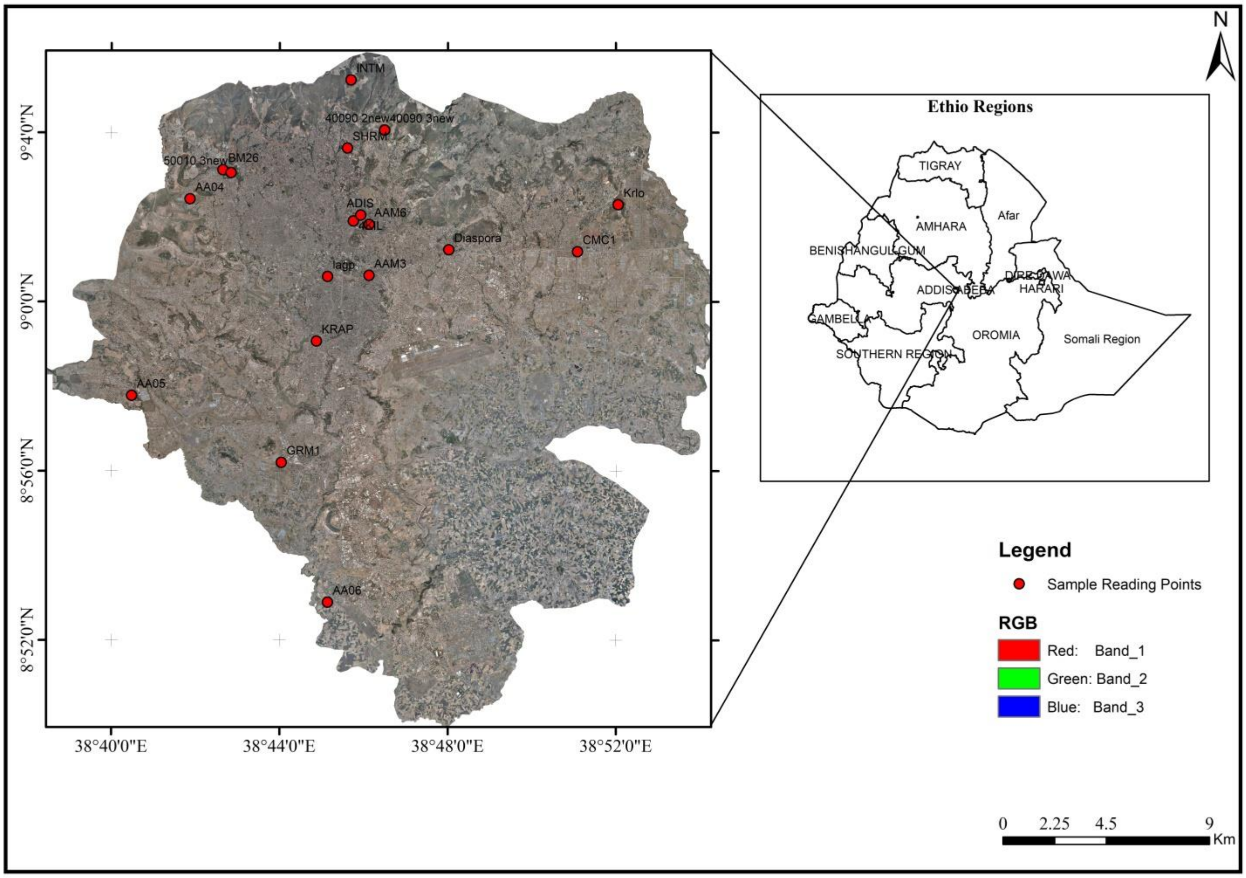

Figure 3-1 Study Area

3.1.2. Topography

Addis Ababa city lies at a minimum altitude of 2015m to a maximum altitude of 3014m above sea level as shown in (Figure 3-2). Because of having the highest altitudinal position, there is a great variation of height within the city. The city has experienced spatial spread mostly towards the southern, eastern, and southwestern parts. The northern part of the city, which is the foothills of the Entoto Mountain which has a higher elevation while the southern part of the city, which is the Akaki-Kality has a lower elevation. The elevation map of the study area is a 20-meter resolution obtained from Geospatial Information Institute Company (GII).

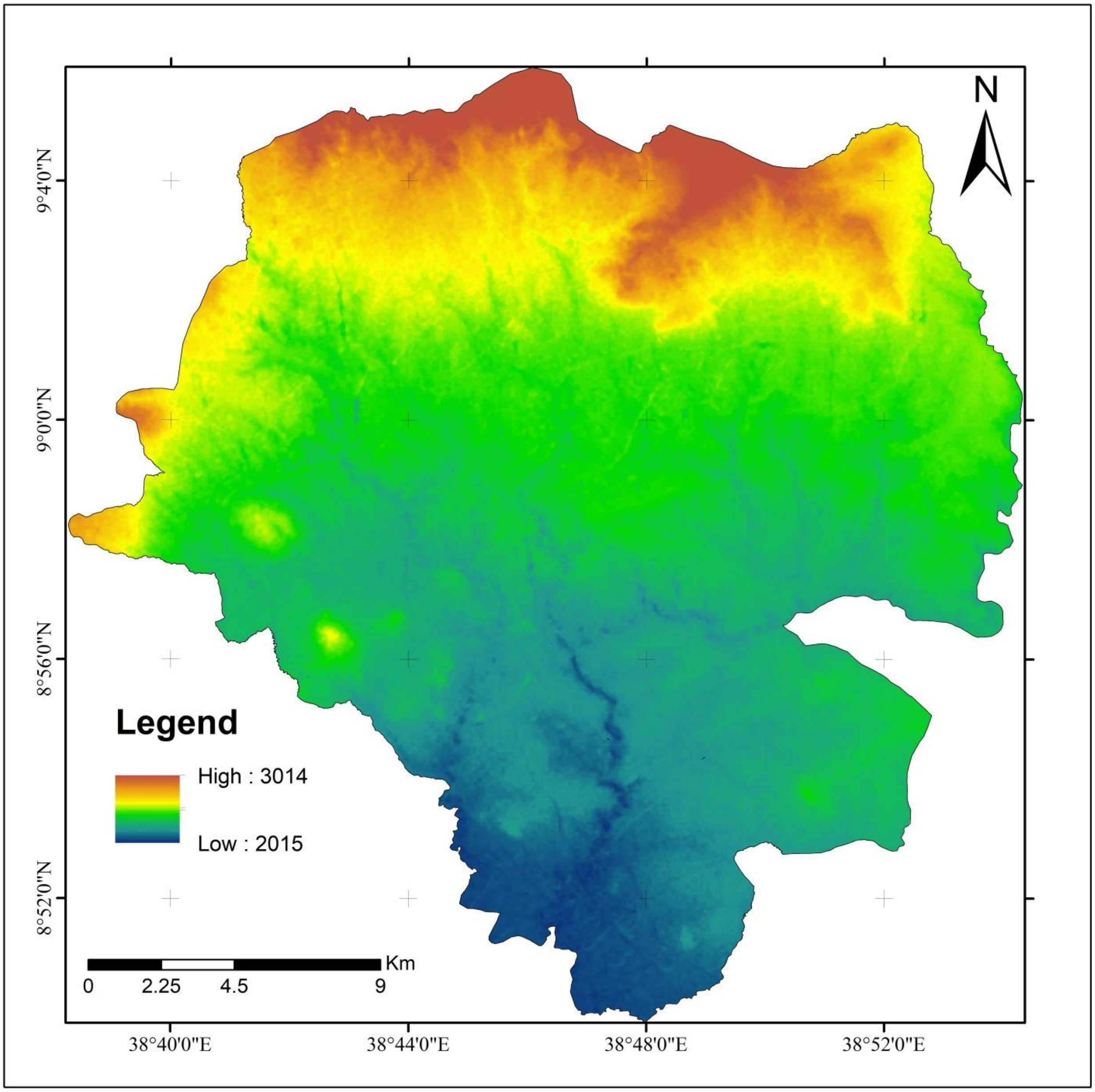

Figure 3-2 Study Area Elevation

3.1.3. Land Use Land Cover

The land is Addis Ababa's most valuable economic resource; the city's land use pattern is characterized by chaotic development that is mostly directed toward horizontal expansion. To analyze the land use and land cover of Addis Ababa city using 25 cm resolution aerial photography data collected in 2017, using the Gregg level 1 classification standard in five different classes for the entire 539km^2 of the city: agriculture accounts for 30.36 percent, forest (green area, riverside greens, etc.) accounts for 9.25 percent, the built-up area accounts for 32.85

percent, water body accounts for 1.75 percent, and bare soil accounts for 25.79 percent. As can be seen in Figure 3-3, the area covered by built-up is quite enormous. As we know, Due to the fastest growth of urbanization in Addis Ababa city through time to time; effective utilization of land, digital land record, and proper utilization of land is required for an effective land administration strategy.

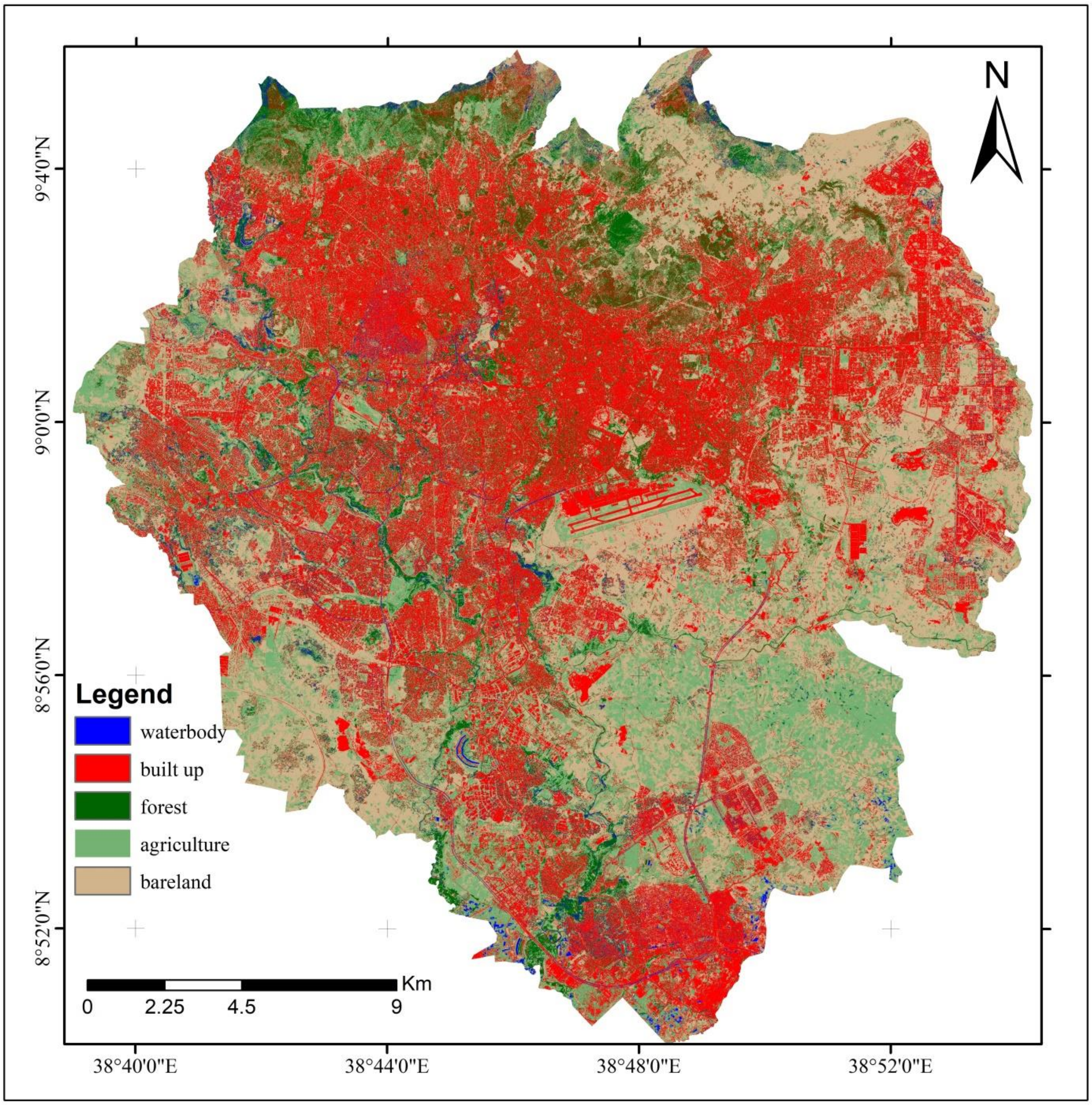

Figure 3-3 Study Area Land Use 2017

3.1.4. Population

According to the Central Statistical Agency of Ethiopia 2007 population census conducted by the Ethiopian national statistics authorities, Addis Ababa has a total population of 3,384,569. However, it is believed that this number was inaccurate when recorded and underestimated the city's population. The city has through recent years seen a robust annual growth rate, and population counts as of 2017 are growing closer to 4 million. The most recent census was scheduled for the 2018 to 2019 fiscal year, as security concerns between 2017 and 2018 delayed it. Addis Ababa is a chartered city and as such, is considered both a city and a state. It is the largest city in the world located in a landlocked country. Per the population recorded at the last census, the city of Addis Ababa has a higher population of female residents than male residents. Almost one-quarter of all people in Ethiopia that live in urban areas live in the capital city.

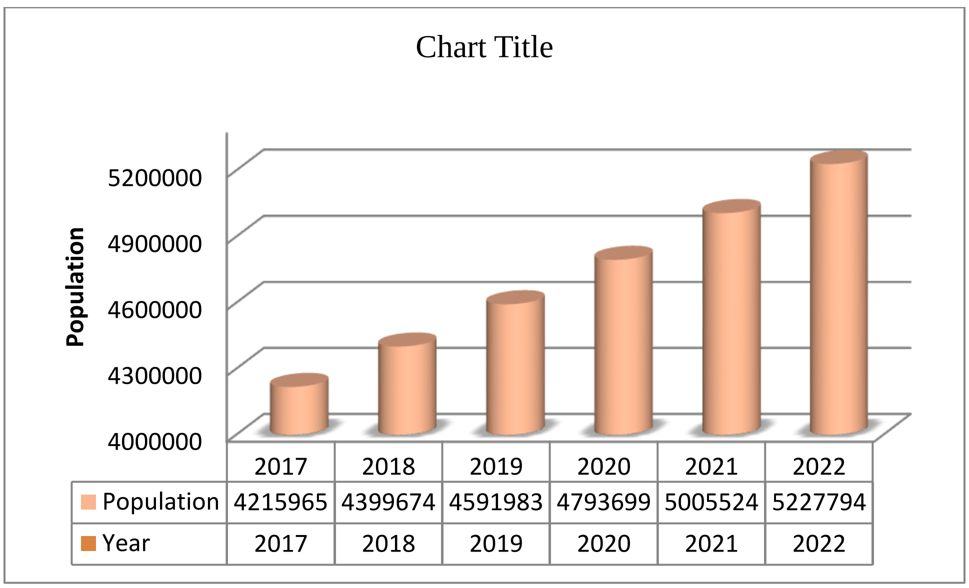

Figure 3-4 Addis Ababa City Population Estimation 2017-2022

3.2. Data

The overall work of scientific research results and findings depends on the quality of the input data. In order to achieve the main and specific objectives of this study, primary and secondary data's were used which include, ground control points, aerial photo, and elevation data. A short description of each dataset is given in this section as shown below.

3.2.1. Primary data source

3.2.1.1. Static GPS Survey Data

The primary data source was used for the evaluation of the horizontal positional accuracy of ground control points and orthophoto using situ measured GCPs. Sample size and spatial distribution of checkpoints play a great role in the accurate evaluation of any geospatial data. Thus, based on ASPRS (2014), NSSDA, and FGDC standards 19 sample points were collected for evaluation of the accuracy of orthophoto and static ground control points in collaborating with Ethiopia geospatial information institute geodetic survey team groups. During this time planning, reconnaissance, field GNSS observation, post-processing, and adjustment were done for the measurement of sample points through DGPS. This provides improved location accuracy; in the range of operations of each system with sub-mm accuracies were done.

Finally, this study tried to an ideal distribution of test point allocation for at least 20 percent being located in each quadrant and test points have been spaced at intervals of at least 10 percent of the diagonal distance across the rectangular data set. The data collection values through the static surveys were listed in (Appendix 2). And the distribution of the GCP looks like in the (Figure 3-1).

3.2.1.2. RTK GPS Survey Data

To evaluate the horizontal positional accuracy of the cadastral parcel corners; independent data having a better accuracy is needed. In this study, a total of 78 coordinate values were collected along with the sharp corners of the fences by collaborating with Ethiopian civil service university students. These points are randomly selected around Yeka sub-city for a pilot test. The coordinates of the sharp corners of the fences were obtained in real-time kinematic global positioning system mode of techniques (RTK GPS). The data collection values through the RTK survey values are listed in (Table 4-3).

3.2.2. Secondary Data Source

Secondary data were collected from different sources including the website, different books, journals, articles, and pieces of literature. The Arial photography for Addis Ababa city was captured by German Company HANSA Luftbild in 2010 at 20cm resolution. Also, in 2017 G.C

Addis Ababa city was captured in 25cm GSD by the former information network security agency (INSA) and the current geospatial information institute (GII). Also, second-order ground control points are collected from Addis Ababa land administration cadastral office.

Table 3-1 Datasets and Source

Data	**Purpose**	**Source**	**Acquisition date**	**Remark**
Orthophoto	To extract point features for assessment	GII	2010 &2017	20 and 25 cm resolution
Cadastral base map	To extract point features from parcel for assessment	Addis Ababa land administration	1997 & 2003	

3.3. Material and Software

In this study, ArcGIS, Track air, Mat lab, Microsoft Excel, INPHO software packages were used for processing, analysis, and interpretations of positional accuracy assessments.

Table 3-2 Material and Software

Software	Purpose	Source	Remark
Track air	To analyze and to assess the mission planning of orthophoto	GII	Licensed
INPHO	To analyze the accuracy of orthophoto	GII	Licensed
ARC GIS	For spatial analysis	Freely available	Free
office 2010	To write, and prepare a thesis work	Freely available	Free

3.4. Methods

In this section, the general methods implemented, applied techniques, and the input datasets used for this study are explained briefly. In order to address the objective of the study the statistical indicators have been used for determination of positional accuracy assessments.

3.5. Positional Accuracy Assessment Procedure for Orthophoto

To validate the accuracy of the orthophoto static field survey were collected. While the static reading of the GPS instrument for a specific point, we take a real-time picture of that point and the surrounding environment as a remark. This is used as a guideline to measure the exact points of GCPs on the orthophoto. (Hasan, 2015) explains that horizontal accuracy is to be assessed using root-mean-squareerror (RMSE) statistics in the horizontal plane, i.e., RMSEx, RMSEy, and RMSEr. To calculate the horizontal root-mean-square error first, the x-coordinate from the reference data is recorded followed by the x-coordinate from the spatial data set being assessed. The root mean square error at x-direction, y-direction, and the horizontal positions were calculated according to the equation (5).

$$RMSE_x = \sqrt{\frac{(\mathrm{Xi} - \mathrm{Xtest})^2}{n}} \tag{5}$$

$$RMSE_y = \sqrt{\frac{(\mathrm{Yi} - \mathrm{Ytest})^2}{n}} \tag{6}$$

Where: x check, i and y check, i are the coordinates of the ith checkpoint in the independent source of higher accuracy, and x test, i, y test, i are the coordinates of the ith test dataset. n is the number of checkpoints tested i is an integer ranging from 1 to n.

$$RMSE_r = \sqrt{RMSE_x^2 + RMSE_y^2} \tag{7}$$

The second way to statistical assess horizontal accuracy using NSSDA multipliers to compute horizontal accuracy at the 95% confidence level, that is 1.7308 times RMSEr. It is assumed that systematic errors have been eliminated as best as possible.

$$\text{Horizontal accuracy } = 1.7308 * \mathrm{RMSEz} \tag{8}$$

And the standard deviation can be calculated through:

$$S_d = \sqrt{\frac{\sum(Xi - Xm)^2}{n-1}} \tag{9}$$

3.6. Positional Accuracy Assessment Procedure for Ground Control Point

In this analysis, before assessing the positional accuracy of the existing second-order GCP data; the independent static DGPS observations were assessed for their local and network accuracies. The processing technique employed with seven-point zero version Leica Geo Office software was capable of generating higher accuracy results. The results were presented as Cartesian Coordinates, referenced to the local coordinate datum. This was performed using a local horizontal plane projection system generated from the reference stations by the transformation method stated in the geospatial information institute parameters. For the reason that, the combined effect of these accuracies might help estimate accuracies between points not directly measured; like parcel maps extracted from orthophoto, or in comparing positions determined from two separate surveys in the future treatment of the parcel map. To compute vertical accuracy using NSSDA at the 95% confidence level, that is multipliers to 1.9600 times RMSEz.

$$\text{Vertical accuracy} = 1.9600 * \text{RMSEz} \tag{10}$$

3.7. Positional Accuracy Assessment Procedure for Cadastral Parcel Corners

To achieve the objective, the sample points are randomly selected from around Yeka sub-city cadastral parcel corners for a pilot test. To statistically assess the accuracy of parcel corners of the fences, twenty-five (25) checkpoints were used in this study. During the collection period, at least four satellites were observed having a PDOP of 3, and also the reading session is from 30 seconds. The base receiver remains stationary over the known point inside the Ethiopian Civil Service University campus on EMA point 102. The base receiver started measurement two hours before the rover receiver starts measurement and stopped measuring after the rover receiver stops. The root mean square error at x-direction, y-direction, and the horizontal positions were calculated according to the equation (5)- (8).

To sum up, in this study, the transformation parameter and positional accuracy assessment method was represented by using statistical measurements. In order to answer the objective of the study, the researcher utilizes statistical analysis to assess the accuracy of transformation parameters, digital orthophoto, second-order ground control points, and cadastral parcels. The analysis presented the residual, mean, and standard deviation in Excel sheet format. Five (5) zero-orders and five (5) first orders, totaling ten (10) ground control points, were used to establish the transformation parameters utilizing the conformal transformation model in different height assumption approaches. Also, nineteen (19) checkpoints collected in static GPS, these check points are used to validate the accuracy of orthophoto and second-order ground control points independently. In addition, 25 check points are collected in RTK survey to validate the accuracy of cadastral parcels. Finally, standard deviation, RMSEz, and LE95% (NSSDA) were computed. The diagrammatic summary of the research methodology is shown in (Figure 3-5).

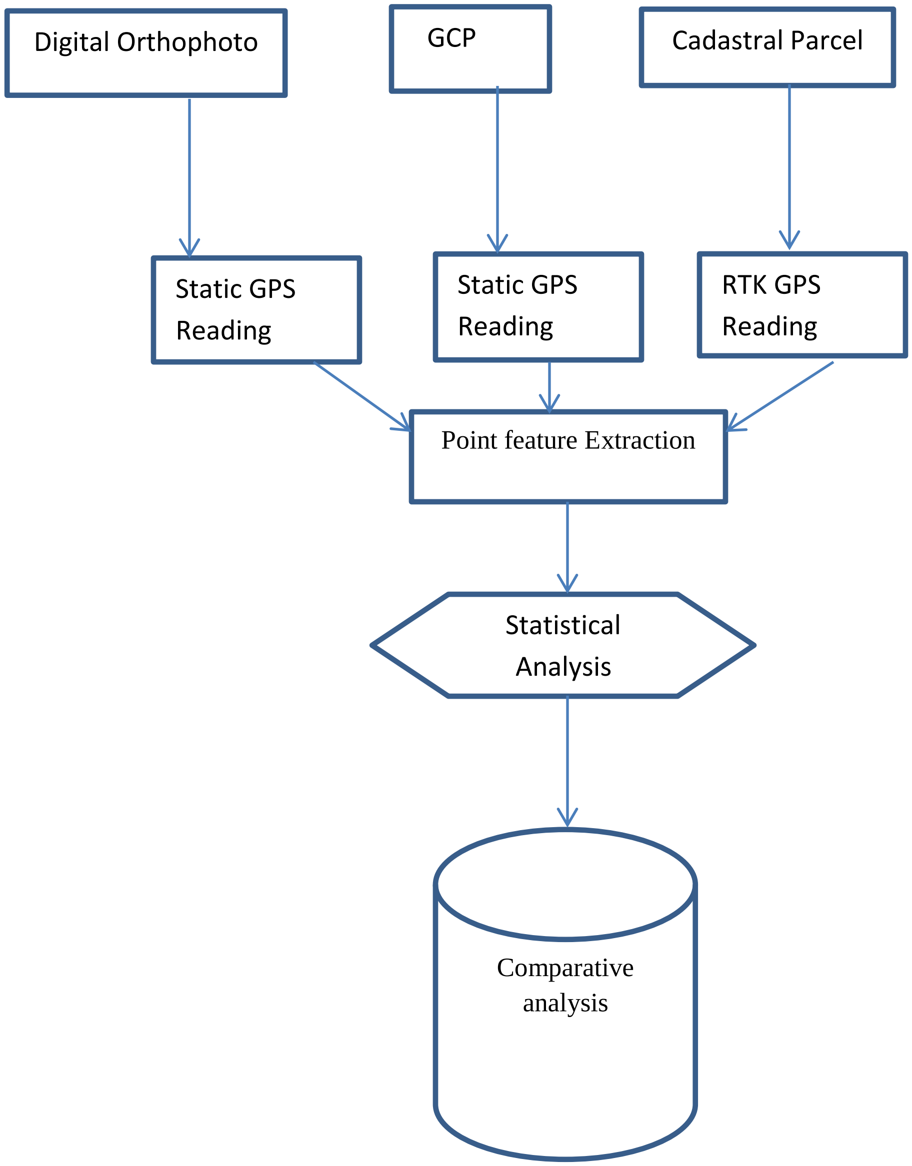

Figure 3-5 Methodological Flow Charts

CHAPTER FOUR

4. Results

4.1. Horizontal Accuracy Assessment Result

This study specifically assesses the horizontal accuracy of orthophoto with the reference to the static GPS survey result. A point-based assessment technique was used to assess orthophoto accuracy. To do these nineteen (19) independent checkpoint coordinates were obtained and each checkpoint was independently compared with the coordinates obtained from orthophoto. The accuracy (RMSE) of the tested data set or orthophoto, as compared to the independent GPS data, is approximately 32.2 cm and 69.5 cm in x and y-direction, respectively as shown in (Table 4-1). The positional accuracy of orthophoto evaluated for 19 checkpoints is estimated as 1.326 meters at a 95 % confidence level.

Table 4-1 Horizontal Accuracy Result Table

	Static GPS Reading		**Residual**			**Orthophoto Generated Points**	
Id	X	Y	X	Y		x1	y1
Diaspora	478066.335	997030.282	0.4033	0.7102	Diaspora	478065.932	997029.572
GRM1	470761.318	987776.177	0.0271	0.7808	GRM1	470761.291	987775.396
CMC1	483669.001	996945.708	-0.0321	0.5717	CMC1	483669.033	996945.136
4KIL	473906.975	998282.67	0.2122	0.6451	4KIL	473906.763	998282.025
SHRM	473650.332	1001459.56	-0.0955	0.333	SHRM	473650.428	1001459.23
Krlo	485447.426	998986.833	-0.6522	0.0812	Krlo	485448.078	998986.751
Lagp	472778.751	995866.296	-0.3652	0.3169	Lagp	472779.116	995865.979
ADIS	474223.525	998537.967	0.1884	0.4476	ADIS	474223.337	998537.519
INTM	473800.787	1004425.06	-0.0351	0.643	INTM	473800.822	1004424.42
BM26	468217.87	1000518.53	0.2223	-0.375	BM26	468217.648	1000518.9
AA05	464237.14	990697.342	-0.4698	0.1545	AA05	464237.609	990697.188
AA04	466786.801	999252.356	-0.5792	0.1735	AA04	466787.38	999252.182
AA06	472774.229	981688.988	-0.3217	-0.0201	AA06	472774.551	981689.008
AAM6	474595.209	998141.239	0.3481	0.4035	AAM06	474594.861	998140.835
AAM3	474588.171	995916.841	0.2751	-0.931	AAM03	474587.896	995917.772
KRAP	472297.746	993059.384	0.3319	0.6926	KRAP	472297.414	993058.691
40090 2new	475275.108	1002245.54	-0.242	-1.169	40090 2new	475275.35	1002246.71
40090 3new	475264.893	1002240.12	-0.273	-1.424	40090 3new	475265.166	1002241.54
50010 3new	468572.963	1000394.95	-0.1	-1.185	50010 3new	468573.063	1000396.13
		Max	0.4033	0.7808			
		Min	-0.6522	-1.424			

	Stdv	0.316454	0.694095
	RMSE	0.322264	0.695533
	RMSEr	0.766564	
	95%	1.326769	

The obtained RMSE and LE95% do not meet the ASPRS 2014 standard at all and they cannot be applied for the production of large-scale planimetric mapping. Similarly, orthophoto does not meet the national standard of Ethiopia for legal cadaster; it states that the overall planimetric RMSE of orthophoto should not exceed 0.30m at the scale of 1:2,000. The residual (errors) between dependent and independent points are between -0.65 up to 0.4 in the x-direction, and -1.24 up to 0.78 meters in y-direction. The residual (errors) for all points are graphically represented in Figure 4-1.

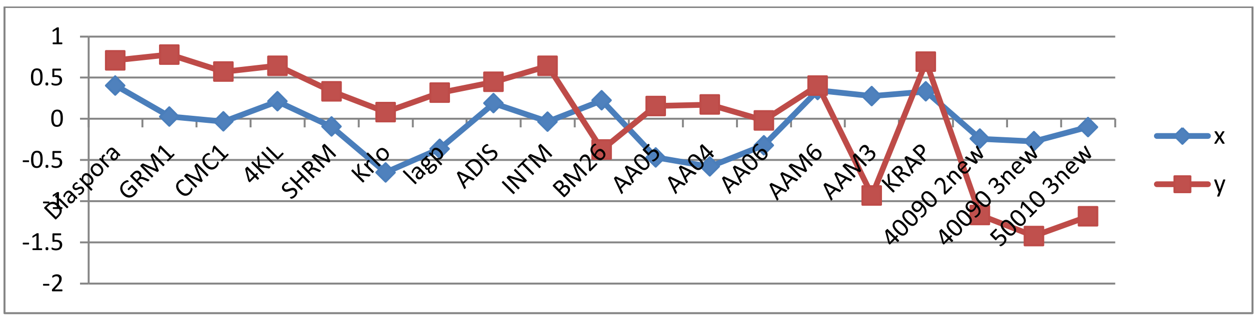

Figure 4-1 Residual Values in Static GPS and Orthophoto

4.2. Ground Control Point Accuracy Result

This study also specifically assesses the accuracy of second-order ground control points that are currently available in the study area with the reference to the static GPS survey. Some of the points were used for aerial triangulation under the orthophoto production stage. Currently, Addis Ababa city uses these second-order control points as a benchmark to expand the block parcels. Because, the current orthophoto which is used as a reference for parcel digitization does not cover all the boundaries of the city, due to the urban expansion of the city beyond its former boundary. A point-based assessment technique was used to assess the second-order ground control point accuracy. 14 independent checkpoints are used to validate the accuracy of second-order points.

The accuracy (RMSE) of the tested data set or second-order points, as compared to the independent GPS data, is approximately 21.25 cm, 17.28 cm, and 52.8 cm in x, y, and z-direction, respectively as shown in (Table 4-2). Specifically, the 4KIL and INTM points are the large shift in both x and y- directions. Because the monuments are relocated from their original position due to the reconstruction of the asphalt road around the place of the monuments exist. These issues are common and frequently happen in Addis Ababa city. Most of the second-order points are damaged, destroyed, and relocated due to some construction of roads, buildings, and other infrastructure in the city. The planimetric positional accuracy of second-order points evaluated for 14 checkpoints is estimated as 0.474 meters at a 95 % confidence level, whereas the vertical planimetric accuracy at a 95% confidence level is estimated as 1.04 meters. Also, the RMSE and LE95% of second-order points obtained does not meet the local and ASPRS 2014 standard at all and they cannot be applied for the production of large-scale planimetric mapping.

Table 4-2 Second Order Points Accuracy Table

	Static Reading						**Second Order Points**		
id	X	Y	H				x1	y1	H1
4KIL	473906.975	998282.67	2433.547	0.577	0.419	1.145	473906.398	998282.251	2432.402
krlo	485447.4256	998986.8325	2597.428	-0.0254	0.0055	0.298	485447.451	998986.827	2597.13
SHRM	473650.332	1001459.56	2567.822	0.253	0.32	0.819	473650.079	1001459.24	2567.003
lagp	472778.7509	995866.2955	2354.358	0.0179	0.0695	0.0495	472778.733	995866.226	2354.308
INTM	473800.787	1004425.063	2949.432	0.456	0.328	1.294	473800.331	1004424.735	2948.138
KRAP	472297.408	993059.034	2283.566	-0.008	-0.005	-0.012	472297.416	993059.039	2283.578
AARA	471239.874	994172.173	2335.555	-0.141	-0.036	-0.083	471240.015	994172.209	2335.638
Salo	473873.444	981865.031	2198.847	-0.022	-0.047	-0.072	473873.466	981865.078	2198.919
GCP1	479644.963	1002218.961	2826.632	0.046	-0.031	0.244	479644.917	1002218.992	2826.388
GCP2	478909.853	990757.076	2286.846	0.009	-0.086	0.141	478909.844	990757.162	2286.705
GCP3	468546.252	1000391.324	2529.617	0.048	-0.034	0.102	468546.204	1000391.358	2529.515
GCP4	486370.907	992772.372	2322.384	0.025	-0.08	0.168	486370.882	992772.452	2322.216
GCP5	479996.058	988763.885	2219.01	0.031	-0.066	0.146	479996.027	988763.951	2218.864
GCP6	473930.301	996680.768	2371.842	0.014	-0.062	0.117	473930.287	996680.83	2371.725
			Max	0.577	0.419	1.816			
			Min	-0.141	-0.086	-0.083			
			Std	0.09678	0.0542	0.941			
			Rmse	0.21256	0.1728	0.5287			
			rmser	0.27394					
			H 95%	0.47413			1.7308		
			V 95%	1.03617			1.96		

The residual errors between dependent and independent points are between -0.14 up to 0.577 meters in the x-direction, -0.086 up to 0.419 meters in y- the direction, and -0.083 up to 1.816

meters in the z-direction. As we have seen in (Figure 4-2) the residual error of the z-direction is comparatively larger than the x and y-direction. The residual errors for all points are graphically represented below in Figure 4-2.

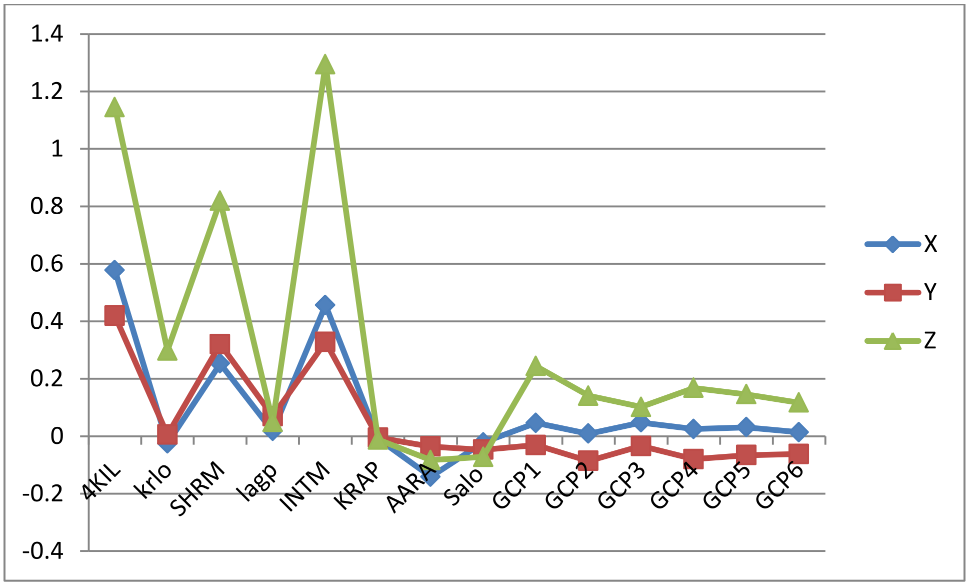

Figure 4-2 Residual Values in Static GPS and Second Order Points

4.3. Accuracy Assessment Based on RTK Survey Data for Parcels Result

Coordinate values obtained using RTK GPS were compared with the corresponding coordinates extracted from the cadastral parcel corner. To do this a point-based assessment technique was used to assess cadastral corner parcels. Statistically, the agreement between the two datasets can be exemplified by analyzing the mean and standard deviation difference between the coordinates from parcel corner and RTK GPS surveying points. The root-mean square error of the point is 0.44 meters and 0.98 meters in x- and y-components, respectively. Whereas, the standard deviation is: 0.441 meters and 0.961 meters in x and y- direction respectively. The overall RMSE of orthophoto is 1.06 meters. In other words, the positional accuracy of orthophoto evaluated for twenty-five checkpoints is estimated as 1.86 meters at a 95 % confidence level through the RTK survey.

Table 4-3 Cadastral Parcel Corner Result Table

Cadastral Parcel Corner		Residual			RTK Points		
X	y	x-residual	y-residual		X	y	
481571.3706	998067.6	0.2826	0.3489	P34	481571.1	998067.2	2425.65
481571.3554	998083.1	0.1564	-0.0718	P35	481571.2	998083.2	2426.489
481571.342	998096.7	0.412	0.0506	P36	481570.9	998096.7	2426.853
481602.4312	998058	-0.3798	0.9491	P73	481602.8	998057.1	2425.196
481571.3259	998113.2	0.5719	0.0264	P37	481570.8	998113.1	2427.986
481599.2429	998115.7	0.0899	0.1349	P45	481599.2	998115.5	2428.662
481627.713	998118.2	-0.284	0.3871	P24	481628	998117.8	2427.463
481639.0235	998044.9	0.3375	-0.0166	P30	481638.7	998044.9	2424.113
481638.3044	998086.5	-0.0976	0.0005	P32	481638.4	998086.5	2425.988
481637.888	998120.1	0.275	-0.2768	P17	481637.6	998120.4	2427.595
481697.8183	998124.3	-0.2557	-0.2361	P7	481698.1	998124.5	2425.285
481700.4606	998089.4	-0.4034	-4.4229	P5	481700.9	998093.8	2423.675
481703.8441	998041.5	-0.4379	0.3141	P1	481704.3	998041.2	2421.352
481567.7468	998120.3	-0.8302	-1.1482	P38	481568.6	998121.5	2428.173
481583.7112	998122.8	-0.4148	-0.3869	P49	481584.1	998123.2	2428.485
481629.6549	998128.3	-0.3851	0.2904	P46	481630	998128	2427.992
481566.1345	998181.8	0.2265	0.5307	P42	481565.9	998181.3	2431.144
481566.0356	998204	0.8376	-0.4442	P44	481565.2	998204.4	2432.215
481625.9664	998206.6	0.0714	-0.3933	P52	481625.9	998207	2431.84
481628.0211	998177.5	-0.7599	-0.3322	P68	481628.8	998177.8	2430.437
481628.9753	998158.1	-0.6517	-0.3686	P70	481629.6	998158.5	2429.16
481638.1024	998128.6	-0.1536	-0.0058	P18	481638.3	998128.6	2427.511
481634.1379	998206.9	0.6899	-0.3381	P76	481633.4	998207.3	2431.817
481666.7296	998130.1	0.1886	0.6797	P21	481666.5	998129.4	2427.099
481663.479	998208.5	-0.442	-0.457	P75	481663.9	998208.9	2431.331
	max	0.8376	0.9491				
	min	-0.8302	-4.4229				
	rmse	0.445119	0.984068				
	Stdv	0.4418	0.961955				
	rmsr	1.080056		1.7308			
	95%	1.869361					

The residual errors between dependent and independent points are between -0.83 up to 0.83 meters in the x-direction, and -0.949 up to -4.42 meters in y- the direction. Also, as we have seen in (Figure 4-3), the residual error of the y-direction is comparatively greater than the x -direction. The residual errors for all points are graphically represented in Figure 4-3.

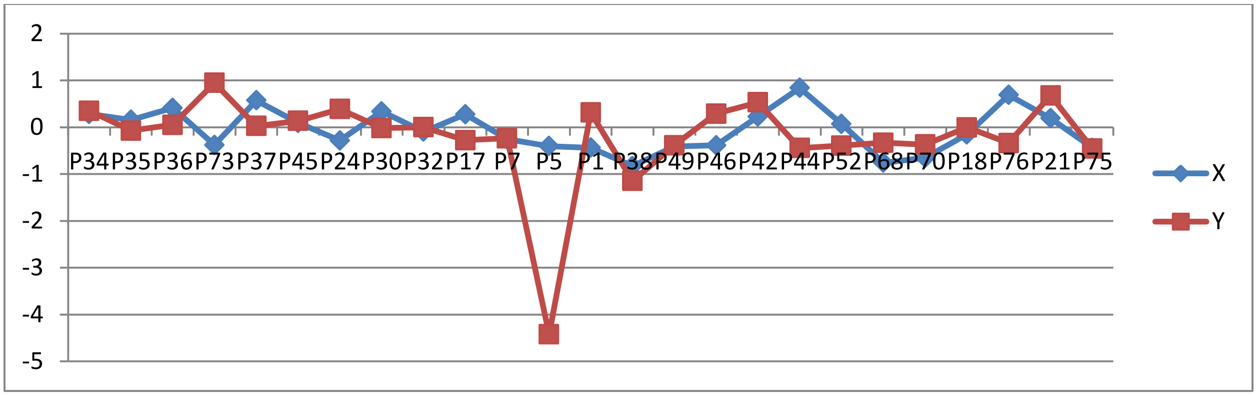

Figure 4-3: Residual Values in RTK Survey Points and Cadastral Parcels.

4.4. Comparative Analysis of Inconsistency of Transformation Parameter Result

In general, three sets of transformation parameters were created for an area of Ethiopia (Parameters used by the National Mapping Agency/Geospatial Information Institute/, NGA for Ethiopia, and NGA-Mean solution). But, users are not sufficiently informed about the uncertainties of transformation parameters and descriptions of transformation models in use. These parameters have been implemented extensively in the present urban cadaster project by incorporating them in their legal frameworks and standards as the mandatory procedure. The same characteristics are used in the geographical data conversions by other local projects and individuals. However, users are becoming increasingly concerned about the accuracy of the transformation parameters and the reliability of the models utilized. Furthermore, many practitioners claim that the uncertainty surrounding the transformation parameters has the greatest impact on the quality of their data. So for this study, the newly developed Molodensky-Badekas transformation parameters in 2019 defined by Aubeker Mohamed are comparatively analyzed with the existing three translation parameters. The easting and northing coordinate of 19 independent static WGS 84 points were converted into the local system using ArcGIS, Global Mapper, and Molodensky-Badekas parameters. The difference in x and y- direction for independent points and the local coordinates (that is converted in to three systems) result as shown in (Table 4-4) – (Table 4-6).

Table 4-4 Comparative Analysis Table1

Independent orthophoto generated points					Static point converted by ArcGIS (X = 165 m, Y =11 m, and Z = -206 m)	
Diaspora	478065.9	997029.572	2.170995	-0.2958	478063.761	997029.8678
GRM1	470761.3	987775.396	2.539505	-0.3769	470758.751	987775.7729
CMC1	483669	996945.136	2.611067	-0.1582	483666.422	996945.2942
4KIL	473906.8	998282.025	2.357549	-0.2301	473904.405	998282.2551
SHRM	473650.4	1001459.23	2.665697	0.0881	473647.762	1001459.142
Krlo	485448.1	998986.751	3.231813	0.3325	485444.846	998986.4185
Lagp	472779.1	995865.979	2.931945	0.0941	472776.184	995865.8849
ADIS	474223.3	998537.519	2.381684	-0.033	474220.955	998537.552
INTM	473800.8	1004424.42	2.604974	-0.2218	473798.217	1004424.642
BM26	468217.6	1000518.9	2.34167	0.7922	468215.306	1000518.108
AA05	464237.6	990697.188	3.028222	0.2509	464234.581	990696.9371
AA04	466787.4	999252.182	3.140034	0.2409	466784.24	999251.9411
AA06	472774.6	981689.008	2.88882	0.4164	472771.662	981688.5916
AAM06	474594.9	998140.835	2.220791	0.0098	474592.64	998140.8252
AAM03	474587.9	995917.772	2.293625	1.341	474585.602	995916.431
KRAP	472297.4	993058.691	2.23554	-0.283	472295.178	993058.974
40090 2new	475275.4	1002246.71	2.811333	1.5913	475272.539	1002245.119
40090 3new	475265.2	1002241.54	2.842329	1.8393	475262.324	1002239.701
50010 3new	468573.1	1000396.13	2.662787	1.5992	468570.4	1000394.531
		RMSE	2.648385	0.786669		
		RMSE(r)	2.762751			
		95%	4.781769			

When the independent static points that are collected in WGS84 datum converted in to local Adindan datum by Arc GIS (X = 165 m, Y =11 m, and Z = -206 m), the RMS error of easting and northing values are 2.65 m and 0.78 m respectively. And the resultant RMS error is 2.76m, as shown in the (Table 4-4).

Table 4-5 Comparative Analysis Table2

Independet orthophoto generated points					Static point converted by global mapper (X = 162m , Y = 12m and Z = -206m)	
Diaspora	478065.9	997029.572	-0.403	-0.71	478066.335	997030.282
GRM1	470761.3	987775.396	-0.027	-0.781	470761.318	987776.177
CMC1	483669	996945.136	0.032	-0.572	483669.001	996945.708
4KIL	473906.8	998282.025	-0.212	-0.645	473906.975	998282.67
SHRM	473650.4	1001459.23	0.096	-0.33	473650.332	1001459.56
KRLO	485448.1	998986.751	0.6524	-0.0815	485447.426	998986.8325
LAGP	472779.1	995865.979	0.3651	-0.3165	472778.751	995866.2955
ADIS	474223.3	998537.519	-0.188	-0.448	474223.525	998537.967
INTM	473800.8	1004424.42	0.035	-0.643	473800.787	1004425.063
BM26	468217.6	1000518.9	-0.222	0.375	468217.87	1000518.525
AA05	464237.6	990697.188	0.4694	-0.154	464237.14	990697.342
AA04	466787.4	999252.182	0.5794	-0.1735	466786.801	999252.3555
AA06	472774.6	981689.008	0.3218	0.0198	472774.229	981688.9882
AAM06	474594.9	998140.835	-0.3479	-0.4036	474595.209	998141.2386
AAM03	474587.9	995917.772	-0.275	0.9306	474588.171	995916.8414
KRAP	472297.4	993058.691	-0.332	-0.693	472297.746	993059.384
40090 2new	475275.4	1002246.71	0.242	1.174	475275.108	1002245.536
40090 3new	475265.2	1002241.54	0.273	1.422	475264.893	1002240.118
50010 3new	468573.1	1000396.13	0.1	1.184	468572.963	1000394.946
		RMSE	0.322219	0.695601		
		RMSE(r)	0.766606			
		95%	1.326842			

When the independent static points that are collected in WGS84 datum converted in to local Adindan datum by global mapper (X = 162m, Y = 12m, and Z = -206m), the RMS error of easting and northing values are 0.32 m and 0.69 m respectively. And the resultant RMS error is 0.766m, as shown in the (Table 4-5).

Table 4-6 Comparative Analysis Table3

Independent orthophoto generated points					Static point of Molodensky (169.674 m, 14.801m, -204.841)	
Diaspora	478065.9	997029.572	2.58	-0.609	478063.352	997030.181
GRM1	470761.3	987775.396	2.949	-0.69	470758.342	987776.086
CMC1	483669	996945.136	3.02	-0.471	483666.013	996945.607
4KIL	473906.8	998282.025	2.767	-0.543	473903.996	998282.568
SHRM	473650.4	1001459.23	3.075	-0.225	473647.353	1001459.455
Krlo	485448.1	998986.751	3.641	0.019	485444.437	998986.732
Lagp	472779.1	995865.979	3.341	-0.219	472775.775	995866.198
ADIS	474223.3	998537.519	2.791	-0.346	474220.546	998537.865
INTM	473800.8	1004424.42	3.014	-0.535	473797.808	1004424.955
BM26	468217.6	1000518.9	2.751	0.479	468214.897	1000518.421
AA05	464237.6	990697.188	3.437	-0.062	464234.172	990697.25
AA04	466787.4	999252.182	3.549	-0.072	466783.831	999252.254
AA06	472774.6	981689.008	3.298	0.103	472771.253	981688.905
AAM06	474594.9	998140.835	2.63	-0.303	474592.231	998141.138
AAM03	474587.9	995917.772	2.703	1.028	474585.193	995916.744
KRAP	472297.4	993058.691	2.645	-0.596	472294.769	993059.287
40090 2new	475275.4	1002246.71	3.22	1.278	475272.13	1002245.432
40090 3new	475265.2	1002241.54	3.251	1.526	475261.915	1002240.014
50010 3new	468573.1	1000396.13	3.072	1.286	468569.991	1000394.844
		RMSE	3.054981	0.697258		
		RMSE(r)	3.133541			
		95%	5.423532			

When the independent static points that are collected in WGS84 datum converted in to local Adindan datum by Molodensky-bedekas seven transformation parameters defined by Abubeker Mohamed in 2019 (169.674 m, 14.801m, -204.841), the RMS error of easting and northing values are 3.055m and 0.697 m respectively. And the resultant RMS error is 3.133m, as shown in the (Table 4-6).

CHAPTER FIVE

5. Discussion

In general, there are three sets of transformation parameters that were defined for a region of Ethiopia: Parameters used by the Geospatial Information Institute, NGA for Ethiopia and NGA-Mean solution (Ameti & Jager, 2016). The Federal Democratic Republic of Ethiopia Geospatial Information Institute uses ΔX = 162 m, ΔY = 12 m and ΔZ = -206 m as an official transformation parameter with the standard deviation and root mean square error of 0.7469m and 2.703m respectively. In addition, NGA (IHO, 2008) published two sets of transformation parameters for Ethiopia (ΔX = 166 m, ΔY = 11 m, and ΔZ = -206 m) with the standard deviation and root mean square error of 0.409 and 0.915, using eight collocated points. The mean solution for Ethiopia and Sudan (ΔX = 166 m, ΔY = 15 m, and ΔZ = -204 m) with the standard deviation and root mean square error of 0.668 and 2.995 respectively using 22 collocated points (IHO, 2008).

In Ethiopia, all the above parameters are inconsistently integrated into different software packages and used as transformation parameters for any geospatial work. Therefore, the positional shift deviates between orthophoto and ground control points from one software package to another package to transform from global to local datum. The finding of the positional shift of orthophoto and ground control points shows that, the inconsistency of the transformation parameters and software packages used.

The obtained Resultant Root Mean Square Error for the orthophoto and ground control point of this study does not meet the ASPRS 2014 standard for legal cadaster preparation at all and it cannot be applied for the production of large-scale planimetric mapping. Consistently the study was carried out in Ethiopia for positional accuracy assessment of three cities, Bahir dar, Debre Markos, and Harer in 2019 with three different scenarios. The study was tested in varying control points, the first scenarios with 10 control points, the second scenarios with 15 control points, and the third scenarios with 20 Ground control points within situ measurements for each city with 12-hour duration of GPS logging time reading. And the data was least-square adjusted in Leica Geo Office(LGO), but all the cities do not meet the local and ASPRS 2014 Standards(Zinabu Getahun et al., 2019).

On the other hand in 2018 the positional accuracy of Bahirdar city was tested independently, the test was carried out in 5 checkpoints with 72-hour durations of GPS reading time. Also, the GPS derived coordinate process in different software packages like GAMIT (GLOBK), Leica Geo-Office, AUSPOS, and APPS with keeping international and national standards. And the result that was obtained in 15 cm ground sample distance orthophoto with 1:2000 scale with AUSPOS processing method meets the national standard limit(Vermeer et al., 2018). The spatial correctness of digital orthophoto and digital line map for Bahir Dar city was also tested in 2017 G.C. Five checkpoints are utilized to assess the horizontal positional accuracy of Bhir Dar city using two processing tools (GAMIT/GLOBK and Leica GeoOfice (LGO)). In addition, 32 coordinate values were read in RTK GPS measurements along the road centerline, and the road center line was digitalized on a digital map (Zinabu Getahun et al., 2017). Also, Zinabu Getahun’s (2017) study satisfies both local and ASPRS 2014 standard criteria. Thus, our study was carried out at ninety checkpoints, while Vermeer's (2018) and Zinabu Getahun's (2017) studies were carried out at 5 checkpoints. As the number of the checkpoints increases the probability of the variation also increases.

In general, the horizontal positional accuracy of orthophoto (x and y), second-order ground control points, and cadastral parcel corners for Addis Ababa city were assessed based on the criteria listed at the national and international level of standards. As it has been seen from the analysis part of this study, the accuracies of the GCPs did comply with the EMA accuracies; as per the results of the analysis. However, the result was not able to meet the accuracy level of ASPRS standards to produce 30cm RMSE of orthophoto (RMSEx (AT) or RMSEy(AT) = ½ * RMSEx(Map) or RMSEy (orthophoto)). Also, individual errors derived from orthophoto might often seem small; but collectively, they can significantly affect data accuracy, impacting how the data can be appropriately used. The finding of this study is in line with Hansa Luftbild study in 2013 for Addis Ababa city, it aimed to assess the positional accuracy of the Ground Control Points and orthophoto, those GCPs were utilized in Aerial photo processing for Addis Ababa City Administration Real Property Registration (Zein et al., 2012).

CHAPTER SIX

6. Conclusion and Recommendations

6.1. Conclusion

Recent advances in geospatial technology have used both global and local reference datum for several applications of works. There are different kinds of geospatial data sources like open source, survey data, photogrammetric data, satellite images, etc. Therefore, before using all these data for any application, they should be in the same datum and must have unique transformation parameters. A wide range of coordinate transformation models is known and available for utilization in geosciences for coordinate transformations between local and global datums. Although the models are known, empirical research on the models and their mutual comparison is valuable because the suitability for utilization generally depends on the accuracy needed and available resources. This study gave greater insight into the empirical accuracy of inconsistency of transformation parameters.

The orthophoto and second-order a ground control point did not meet the local EMA standards and APRS 2013 standards to satisfy the national standard of urban cadaster in Addis Ababa city, according to the data used, the methodology, and having the outcome that indicates. Another key finding of this study is that the horizontal accuracy of parcel corners utilized for cadastral mapping, which is now officially used by Addis Ababa land admiration and urban cadaster office, exceeds the local and ASPRS 2014 accuracy criteria.

Generally, in our analysis; even if, the accuracy of the GCPs did not meet the ASPRS positional accuracy level, it could still definitely assure attain a minimum of 30 cm RMS error, which is within an acceptable range of error to be committed as per the local standard. But, as the analysis of the test points of the field check has demonstrated, a representative RMS error of 1.326m has been committed. This forced the analysis to conclude that the orthophoto map produced did not meet the accuracy requirement of the local standard and legally defined parcel map extraction. Finally, the software packages used to convert from global to local reference datum differ from one another, causing major changes in the values of specific points. For example, if we use (X = 162m, Y = 12m, and Z = -206m) to translate coordinates values from global to local; the RMS error of easting and northing values are 0.32 m and 0.69 m respectively. When we translate coordinate by using ArcGIS software(X = 165 m, Y =11 m, and Z = -206 m), the RMS error of easting and northing values are 2.65 m and 0.78 m respectively. And, if we apply the Molodensky-Badekas transformation parameters defined by Abubeker Mohamed in 2019 (169.674 m in X, 14.801m in y, and -204.841 in Z) , the RMS error of easting and northing values are 3.055m and 0.697 m respectively. The shift between the newly transformed model and the existing one indicates that even if, the accuracy of the parameter varies; there is no significant effect on the positional accuracy of the spatial data, until we are using unique, accurate, and common transformation parameters for each software package to convert from global to local datum.

6.2. Recommendations

Based on the outcomes of this study: the following recommendations have been provided for practical future work and future researchers:

- Rather than using the default transformation settings included in various tools, geospatial scholars should create customize transformation parameters. The government must also provide standards and procedures for converting from a global to a local reference datum. To avoid inconsistency and confusion, the researchers perform extensive empirical investigations on this topic and share their findings with geospatial scholars, organizations, and software developers. In addition to this, the feature works shall be concentrated on the densification of static ground control points that are used for validating the accuracy of orthophoto.

- While transforming raw data taken by GPS; one has to search for the best fitting transformation parameter. If there is a variation in transformation parameters in different software packages then there is a significant effect on the quality and integration of spatial data. The concerned body would take a responsibility to precisely define the local ellipsoid. If this might not be possible; there should be enough evenly distributed GCPs. In addition, because many control points have vanished or been damaged as a result of urban growth and road construction, the Ethiopian Geospatial Institute should re-construct the points and protect them from damage.

- According to the results of the study, it is recommended to use static GNSS data as a reference rather than RTK GNSS data when evaluating accuracy assessment. Feature research should concentrate on the transformation parameter effect on cadaster to bring good governance and minimized land-related conflicts.

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Appendix 1 **Field Survey Photo**

Appendix 2 Static GPS Reading

	Static GPS Reading			Positional Uncertainty (95% C.L.)			Duration (Hours)
				East(m)	North (m)	Height(m)	
id	X	Y	H	0.012	0.009	0.029	4:04
Diaspora	478066.335	997030.282	2393.943	0.022	0.09	0.043	2:54
GRM1	470761.318	987776.177	2218.159	0.01	0.008	0.026	4:38
CMC1	483669.001	996945.708	2388.294	0.013	0.01	0.034	4:02
4KIL	473906.975	998282.67	2433.547	0.345	0.06	0.326	2:30
SHRM	473650.332	1001459.56	2567.822	0.015	0.009	0.043	3:22
Krlo	485447.426	998986.833	2597.428	0.009	0.006	0.015	3:22
lagp	472778.751	995866.296	2354.358	0.037	0.019	0.113	2:03
ADIS	474223.525	998537.967	2446.207	0.036	0.036	0.013	2:53
INTM	473800.787	1004425.06	2949.432	0.0032	0.001	0.0075	2:01
BM26	468217.87	1000518.53	2534.646	0.0040	0.0029	0.0087	2:10
AA05	464237.14	990697.342	2287.091	0.0033	0.0030	0.0104	2:15
AA04	466786.801	999252.356	2451.103	0.0045	0.0039	0.0111	2:05
AA06	472774.229	981688.988	2075.592	0.0035	0.0031	0.0095	2:35
AAM6	474595.209	998141.239	2420.622	0.0057	0.0050	0.0141	2:14
AAM3	474588.171	995916.841	2337.629	0.0063	0.0051	0.0152	2:31
KRAP	472297.746	993059.384	2284.659	0.0077	0.0072	0.0214	2:30
40090 2new	475275.108	1002245.54	2596.64	0.0127	0.0165	0.0037	2:29
40090 3new	475264.893	1002240.12	2589.532	0.0063	0.0057	0.0177	2:33
50010 3new	468572.963	1000394.95	2529.617	0.012	0.009	0.029	4:04