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Addis Ababa Institute of Technology

School of Civil and Environmental Engineering

GEOMATICS AND GEODESY PROGRAM

POSITIONAL ACCURACY ASSESSMENT OF ORTHOPHOTO AND DIGITAL ELEVATION MODEL: A CASE STUDY IN DIRE DAWA CITY, ETHIOPIA

By:

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September 2021

Addis Ababa, Ethiopia

Approval Sheet

Addis Ababa Institute of Technology

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The undersigned have examined the thesis entitled 'positional accuracy assessment of digital orthophoto and its elevation model in the case of Dire Dawa city.' presented by Nathanael Merkin, a candidate for the degree of master of science, and hereby certify that it is worthy of acceptance.

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As a Master's research advisor, I certify that this Masters of Science (MSc) thesis was evaluated and prepared under my guidance.

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Abstract

The quality of any geospatial data depends on its accuracy for mapping, design, analysis, and decision-making purposes. In this study, positional accuracy assessment was carried out with two datasets orthophoto and digital elevation model (DEM) generated from an aerial survey having 15cm ground sample distance using statistical analysis. Seven checkpoints were measured in the field using a static differential global positioning system technique Trimble 5800 global positioning system (GPS) and processed with Leica Geo-office (LGO) software and different online post-processing. Also, fifty-two parcel corners and road centerlines were selected and measured with real-time kinematic techniques for feature comparisons. The Easting and Northing coordinates from these points were compared with their corresponding positions extracted from orthophoto for horizontal accuracy. Seventy checkpoints were compared with their corresponding positions extracted from DEM for vertical accuracy.

The accuracy of orthophoto evaluated with Static GPS measurement was root mean error (RMSE) of 12.90cm in Easting and 18.40cm in Northing and positional error of 22.5cm having horizontal accuracy of 38.9cm at 95% confidence level. The GPS coordinates of the parcel corner deviate from the corresponding coordinates of the orthophoto parcel corner having 26cm in the x-direction and 15cm in the y-direction. Therefore, it can be concluded that the accuracy of orthophoto map is within the national standard of error budget and has been used for different applications. The vertical accuracy results in RMSE of 4.06m, which shows that it does not meets the national standard of vertical data accuracy requirement, i.e., root mean error in z- direction (RMSEz) of ± 0.45m and American standards of photogrammetry and remote sensing (ASPRS) standard. Elevation deviation increases with slope increases from flat terrain characteristics to escarpment terrain characteristics.

ACKNOWLEDGMENTS

I would like to express my heart full thanks and appreciation to my advisor, **Dr. Tulu Besha** (the president of Ethiopian geospatial information institute) for his interest in my research work, and his usual advice, guidance, and encouragement to accomplish my thesis work successfully. My sincere thanks go to my colleagues Mr. Epheson Kastro and Mr. Mekuanint, for their valuable constant encouragement and support during all phases of this work. I am also thankful to the Dire Dawa University, geospatial information institute and Dire Dawa land development and management offices, and the integrated land information project office for their cooperation in providing me all kinds of information and material support. Lastly, my special thank is to my lovely wife Gizework Alemayehu for her encouragement to succeed in my study.

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LIST OF ABBREVIATIONS

APPS	Automatic Precise Positioning Service
ASPRS	American Society for Photogrammetry and Remote Sensing
AUSPOS	Australian positioning service
CORS	Continuously Operating Reference Stations
DGPS	Differential global positioning system
EGII	Ethiopia geospatial information Institute
CSRS-PPP	Canadian Spatial Reference System-Precise Point Positioning
EGM	Earth gravity model
ESRI	Environmental Systems Research Institute
FGDC	Federal Geographic Data Committee
GCP	Ground control point
GIS	Geographical information system
GNSS	Global navigation satellite system
GPS	Global positioning system
IGS	International GPS Service
INSA	Information Network Security Agency
OPUS	Online positioning user service
RMSE	Root-mean-square error
GSD	Ground Sample Distance
RMSDz	Root-mean-square-difference in elevation (z)
ITRF	International terrestrial reference frame
LGO	Leica geo-office
NASA	National Aeronautics and Space Administration
NSSDA	National standard of spatial data accuracy
RMSEr	Horizontal linear RMSE in the radial direction
RMSEx	Horizontal linear RMSE in the X direction (Easting)
RMSEy	Horizontal linear RMSE in the Y direction (Northing)
RMSEz	Vertical linear RMSE in the Z direction (Elevation)
RTK	Real Time Kinematic
RTX	Real-Time extended

CHAPTER 1 INTRODUCTION

1.1 Background of the Study

The positional accuracy assessment of geospatial data such as digital orthophoto maps and DEM is important for defining the applicability of the datasets requiring different levels of accuracies. Boye et al., (2016) defined the positional accuracy assessment as the important method used to evaluate the quality of spatial datasets. Assessing the positional accuracy is a critical issue in implementing consistency in spatial data accuracy. To determine the accuracy level of spatial data for the specific purpose the data should be assessed according to different international and national standards.

International organization for standardization defines five elements of data quality: completeness; logical consistency; positional; temporal and thematic. The last three elements are based on accuracy and this thesis is concerned with positional accuracy. The most common errors in digital geospatial data sets are positional error, thematic or attribute error, inconsistency, and incompleteness, of which the positional and attribute errors are considered the most important. This thesis focuses on the assessment of positional accuracy. As spatial data transfer standard (ICA, 1991) describes three methods for assessing positional accuracy, these are deductive estimate, internal evidence, and comparison to source. Positional accuracy assessment methods used in these studies were based on comparison because of enough checkpoints having higher accuracy standards.

Different professionals perform the accuracy assessment for different reasons due to the desire to know how bad or good a map or the spatial data is made. The others need or want to increase the quality of the map information by identifying and correcting the sources of errors. Assessing the Positional accuracy is about estimating the closeness of all positions on a map or data layer, in these cases georeferenced orthophoto, matches or fits the corresponding positions of features they represent on the ground in the desired projection system (i.e., a frame of reference). Positional accuracy assessment determines how closely the positions of discrete objects or features are compared to their actual locations on the ground. The quality of digital ortho imagery, digital planimetric data, digital elevation data, and the like spatial dataset is determined as well. The data layers to be tested are compared with the reference datasets that are more accurate than the corresponding ones. When we talk about the position of features or objects we talk about three components Easting, Northing, and height. In other words, assessing positional accuracy means assessing both horizontal and vertical components. When positional accuracy assessment is required, horizontal accuracy shall be assessed by comparing the coordinates of test points in the dataset with corresponding coordinates from an independent source of higher accuracy. Vertical accuracy shall be assessed by comparing the elevations of test data represented by the dataset with elevations from an independent source of higher accuracy (FGDC, 1998). This is done by comparing the elevations of the checkpoints with elevations from the dataset at the same x/y coordinates (ASPRS, 2015).

Horizontal accuracy shall be assessed by comparing the coordinates of checkpoints in the dataset with coordinates of the same points from an independent source of higher accuracy (FGDC, 1998). In general, the positional accuracy of orthophoto refers to the accuracy at which the position coordinates (Easting, Northing, and ellipsoidal height) of the spatial objects that are well recognized on the orthophoto are estimated about their corresponding ground-truth coordinates acquired at the same locations using an independent ground-based in-situ measurement such as global navigation satellite system (GNSS) observations (Congalton & Green, 2009). In theory, the accuracy of ground-based reference data used to validate the accuracy of photogrammetric data shall be at least three times more accurate than the photogrammetric data being assessed (Congalton & Green, 2009), while the error in positional accuracy of photogrammetric data is the difference between the coordinates of the same selected spatial objects as acquired from the photogrammetric survey when compared to coordinates measured using GNSS receivers Zinabu et al., (2017). The study will accomplish a statistical and testing methodology for estimating the positional accuracy of digital orthophoto and digital elevations model against the in-situ measurement of higher accuracy (NSSDA, 1998).

1.2 Statement of the Problem

The positional accuracy assessment of geospatial data can be affected by several factors measurement errors as well as errors faced with data processing. For example, weather conditions such as snow or cloud might give a false representation of the ground and will affect the quality of the photogrammetric products. The urban area that is usually hidden by structures such as tall buildings, tree canopies, and vegetation cannot be accurately mapped (Sharlene pillay, 2015).

Raw aerial photography or digital imagery has large geometric distortion that is caused by various systematic and nonsystematic factors and completely shifts and mismatches features of real earth on the ground. Several factors can affect the positional accuracy of a map or georeferenced image. For example, the sensor lens may be distorted, or the aircraft carrying the sensor may suddenly tilt or yaw, changing the relationship of the sensor's image plane to the ground (Congalton & Green, 2009). To overcome the challenges regarding the geometry of features to align with the reality on the ground the optimal number and spatial distribution of ground control point (GCP) should be used in the process of geo-referencing the imagery. This is why geometric corrections, as preprocessing operations, are normally required before imagery analysis and extraction of information. But this does not mean georeferenced imagery is accurate. Therefore, it is necessary to check the accuracy of orthophoto and other derived products for specific applications. Because small geometric shift or distortion on orthophoto and DEM can lead to a large shift in position on the ground surface.

Therefore, the expert or the analyst must be sure about the accuracy both horizontally and vertically. Producing a photo-quality digital image of surface features in their geometrically corrected, true map projection, tying each pixel in an image to its true earth location, the user is enabled to make photo interpretation decisions, allowing for better analysis and delineations. Most of the time in the country many projects have been gone but in the end, the accuracy level does not meet international as well as national map accuracy standards. If the output or produced orthophoto having less accuracy, this will enable wrong interpretation and extraction of information and finally wrong decision. Now, a day dependency on these digital datasets has been increased but no adequate studies are conducted to assess the positional accuracy of geospatial data. In this regard, Orthophoto produced should be subjected to rigorous positional accuracy assessment to enhance the accuracy and reliability of the data for decision making. The study will evaluate the accuracy of orthophoto and DEM covering the region of Dire Dawa city that help us to understand the accuracy level of the products.

1.3 Objectives of the Study

1.3.1 General Objectives

The general objective of the study is to assess the positional accuracy of the digital orthophoto map and digital elevation model of Dire Dawa city.

1.3.2 Specific Objectives

- To assess the horizontal accuracy of the orthophoto.
- To evaluate the vertical accuracy of the DEM.
- To assess the dependency of positional accuracy on terrain characteristics.

1.4 Significance of the Study

Digital orthophoto and DEM have many applications in the area of mapping, analysis, and planning. They are extensively used for urban and rural cadaster to mitigate land-related problems as well. The government of Ethiopia is committed to establishing a good land governance system. Many international groups such as UK and Finland are supporting the land registration program. Land investment for transformation and Rewa projects can be mentioned as an example (Binyam , 2015).

Accurate orthophoto and the DEM data will be used for different analyses directly related to the decision-making processes like land management, land banking, cadaster, and engineering applications. To solve land and land-related problems especially for cadastral purposes the positional accuracy shall meet national standards. The valid accuracy is not only useful but must be required to establish a legal cadaster for good governance in Ethiopia (Congalton & Green, 2009) that may be a great solution for implementing land administration problems. Assessing the positional accuracy is a critical issue in implementing consistency in spatial data accuracy.

1.5 Scope of the Study

This study addresses both horizontal and vertical positional accuracy assessment of georeferenced orthophoto and DEM generated from aerial photos of Dire Dawa city, respectively.

1.6 Limitation of the Study

The distribution of checkpoints used in this study to evaluate the positional accuracy of the orthophoto and DEM does not meet the NSSDA standard of the distance between checkpoints (d/10) rule that is due to inappropriate ground surfaces. The in-situ measurements are not connected to national networks of zero or first-order geodetic benchmarks of the area but with second-order established ground control points. The number checkpoint for static survey not meets, and distribution checkpoints for real-time survey do not meet ASPRS standard. The number of checkpoints used on different terrain characteristics to evaluate the dependency on positional accuracy was not equal or the same. And having old batteries of instrument the static field measurement did not take for more than four hours.

1.7 Organization of the Study

This thesis was organized into five chapters. The first chapter includes a detailed discussion of the background; statement of the problem; objectives of the research and significance of the study is put in this subdivision. Chapter two presents relevant literature reviewed to the study. Chapter three deals with the description of the study area, data sources, material, and methodology. Chapter four deals with results and discussion and finally chapter five deals with conclusion and recommendation.

CHAPTER 2 LITERATURE REVIEW

2.1 Positional Accuracy Assessment

Positional accuracy assessment is the combination of three words that are position, accuracy, and assessment. The position is about the Easting, Northing, and height of features on, above, or below the earth's surface. Accuracy is the closeness of measurement or observations with the true value which means, in this case, it refers to how aerially sensed photogrammetric end products (orthophoto and DEM) data is closed to what is actually on the ground truth data. The assessment is the way of determining the accuracy or quality of measured and ground truth value. Accuracy can be classified as thematic accuracy and spatial accuracy. This study is about spatial accuracy assessment of the end products of photogrammetric data by quantifying errors in the locations selected checkpoints. Different works of literature define accuracy assessment in different ways but they have the same concepts. For example, Congalton (2001) defines accuracy assessment as a key component of any project employing spatial data. Anupam (2017) defines accuracy assessment as 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. Humboldt State University (2019) expresses the 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. Usually, this data is known as ground-truth data that has been collected in the field. Esri (2020) defines positional accuracy as the quantifiable value that represents the positional difference between two geospatial layers or between a geospatial layer and reality. These assessments can either be qualitative or quantitative. Qualitative is usually a quick comparison to see if the aerially sensed data and corresponds to what is on the ground whereas quantitative assessments attempt to identify and quantify error. In these assessments, we compare map data with a reference of ground truth data. Accuracy is important because remotely sensed data are often used for mapping and developing environmental models that are used for management and decision-making purposes.

But why do we assess the accuracy of spatial data, different kinds of literature list many reasons for it? For example, Congalton & Green (2009) lists three reasons, the first is a desire to know how good data you have made for the sake of satisfaction. The second is to increase the quality of the data by identifying and correcting the sources of errors. Third, to compare various related data sets to test which is best. Also, Congalton (2001) explains the importance of accurate assessment as to know how well you are doing and to learn from your mistakes, to quantitatively compare methods, and to use the information resulting from your spatial data analysis in some decision-making process. Same as the third reason mentioned, this study also wants to compare two data sets and to decide the accuracy level that will help the end-users for decision-making purposes. Positional accuracy determines how closely the position of spatial features shown on the orthophoto map is with the true position on the ground. Measuring physical dimensions of features on or above the surface of the earth through the aerial survey was obvious and has much more advantages than ground surveying. The airborne sensor will record the features or objects in the form of the image in digital form and stores them in pixel format. After passing through long processes in the office at the end it produces the coordinates of a particular point, a planimetric feature, and a graphic representation of the terrain (DEM). However, identification of planimetric features and elevations derived from photogrammetry are less accurate than ground surveys (when compared to conventional or GPS ground survey methods using appropriate elevation procedures). Small geometric shift or distortion on end products of photogrammetry leads large shift in the position on the ground surface. Therefore, positional accuracy level according to user requirement and application should be assessed whether it meets the required standard or not. ASPRS defines horizontal accuracy as the root mean square (RMS) error in terms of planimetric survey coordinates (X, Y) for checked points as determined at the full ground scale of the map and vertical accuracy as RMS error in evaluation in terms of evaluation datum for well-defined points only (ASPRS, 1990).

The NSSDA implements a statistical and testing methodology for estimating the positional accuracy of points in digital geospatial data, concerning georeferenced ground positions of higher accuracy (NSSDA, 1998). According to the NSSDA standard, the horizontal accuracy shall be tested by comparing the planimetric coordinates of well-defined points in the dataset with coordinates of the same points from an independent source of higher accuracy. Vertical accuracy shall be tested by comparing the elevations in the dataset with elevations of the same points as determined from an independent source of higher accuracy. A well-defined point represents a feature for which the horizontal position is known to have a high degree of accuracy and position to the geodetic datum. For accuracy testing, well-defined points must be easily visible or recoverable on the ground, on the independent source of higher accuracy, and on the product itself. NSSDA implements a statistical and testing methodology for estimating the positional accuracy of points on maps and in digital geospatial data, to georeferenced ground positions of higher accuracy and recommends accuracy testing by an independent source of higher accuracy is the preferred test for positional accuracy. The independent source of higher accuracy shall be of the highest accuracy feasible and practicable to evaluate the accuracy of the dataset.

Positional accuracy is also divided into two these are horizontal and vertical accuracy. Horizontal accuracy is the horizontal (radial) component of the positional accuracy of a data set to a horizontal datum, at a specified confidence level. Thus, the establishment of the standards is critical in implementing consistency in spatial data accuracy. Therefore in the united states of America, the two well-known that is ASPRS and FGDC set standards and implements a statistical and testing methodology for estimating the positional accuracy of points on maps and in digital geospatial data, to georeferenced ground positions of higher accuracy (FGDC, 1998).

The accuracy of a digital orthophoto map is very important especially when it is used in the creation or updating of the GIS database by digitizing (Svetoslav, 2015). The researcher tried to consider in two ways, by comparing the location of pixel elements to their true location on the face of the Earth and the second is by considering pictorial defects and tonal differences, both within and across the ortho map sheets. Some factors affect the quality of DEM data. These factors are the accuracy of data source, sampling density, terrain roughness, grid resolution (pixel size), interpolation source (Shingare & Kale, 2013). The study area has an automatically changing terrain type at the side center of the city it represents characteristics of the surface such as flat, hilly, mountainous, etc.

2.2 Orthophoto Map

The aerial photograph is simply a photograph taken using aerial cameras mounted on the aerial vehicle. Before taking the photos a camera has been calibrated for specific purposes. It is necessary to calibrate the camera, to relate image coordinates in a camera-bound system with real-world coordinates in the terrain (Martin. & Getachew., 2018). Robert et al., (1997) proved that an aerial photograph contains many distortions due to the lens, the camera's attitude, and the shape of the earth. All those distortions should be corrected through different software processing techniques to remove the effects of distortions that allow end-users to generate products that may appear as ortho-rectified images (orthophoto map). Accurately generated digital images have constant positional accuracy throughout the image. However, the techniques used to generate many of these products may not produce an accurate digital image in position. An orthophoto is an aerial image that has been geometrically corrected so uniform throughout the image. Now a day's software applies the ortho-rectification process to create an image with constant distance measurement across the entire image. The terrain effects and distortions from the camera's lens and other sources were corrected to generate an accurate orthophoto map. A digital orthophoto typically has a geographic reference to the Earth, such as a UTM or State Plane coordinates, so each pixel in the photo can be accurately located.

2.3 Digital Elevation Model

DEM is simply the generic name to represent a spatially referenced data set of geographical distribution by modeling. DEM is a digital representation of terrain relief, describing the geometry of the ground surface. It consists of a set of coordinates and an interpolation algorithm to give a continuous coordinate designation to every point on the ground surface (Mark & Tina, 2013). Height information having a better quality of the earth's surface is essential for many applications. Since the topography of the earth's surface varies throughout the area and representing ground features from above at a far distance is not an easy task. DEM is normally generated through the following steps data acquisition step Li et al., (2006), grid space resampling step, the height of a point interpolation step and finally DEM is represented. An error of generated DEMs can be introduced throughout the above steps. The quality of DEM is affected by several factors, such as sensor types, algorithm, terrain type, grid spacing, and characteristics (Hebeler & Purves, 2009).

2.4 Datum

2.4.1 Horizontal Datum

One commonly used datum to describe the entire earth is WGS 1984 which refers to the World Geodetic System of 1984. WGS84 datum has been refined on several occasions and is now aligned with the international terrestrial reference frame (ITRF) to within a few centimeters worldwide. The Global Positioning System uses the WGS84 as its reference system. The data used for this study were Orthophoto and DEMs both are defined concerning the WGS84 horizontal datum. Since the datum being is global it was transformed to local datum by using different transformation parameters. Continentally, Ethiopia is found in East Africa. There is a datum that best fits East African countries, named Adindan. There is no defined ellipse by the name of Adindan, thus it is referenced to the ellipsoidal system of Clarke 1880. Therefore the local datum transformation is made using the ellipsoids of WGS84 and Clarke 1880. To make the measurements more precise and accurate regionally, a local transformation parameter to each region is needed. Accordingly, Ethiopia has developed its transformation parameter from the datum of WGS84 to Adindan (Table-2.1), concerning Clarke 1880 ellipsoid (FDRE, 2003). During the processing, -165m, -12m, and + 206m were used as transformation parameters in x, y, and z respectively.

Table 2.1: Transformation parameters from WGS84 to Adindan

Reference Ellipsoid	**Transformation Parameters**		
	ΔX	ΔY	ΔZ
Clarke 1880	-165m ± 3m	-11m ± 3m	+ 206m ± 3m

2.4.2 Vertical Datum

The set of fundamental elevations are referenced with other elevations. To perform a statistical comparison between elevation models and ground truth data, ground-based ellipsoidal heights data were transformed to orthometric heights defined regarding the

EGM1996 geoid model. The geoid model contributes to the vertical component of the reference system so that ellipsoidal GNSS heights can be converted to orthometric elevations for practical uses. His real challenge lies in knowing the relationship between the ellipsoid and the geoid. Once we determine the difference between these two surfaces, called the "geoid-ellipsoid separation" or "geoidal height", at a given point, we can then apply the geoidal height to our GNSS height measurement to get the mean sea level elevation. The ellipsoidal height is measured orthogonal to the ellipsoid whereas the orthometric height is measured orthogonal to the geoid, orthogonality is with ground points on the earth's surface. When we talk about the height information of the georeferenced data set we remember the height system or vertical datum. Figure 2.1.1 below shows different height systems with their vertical datum.

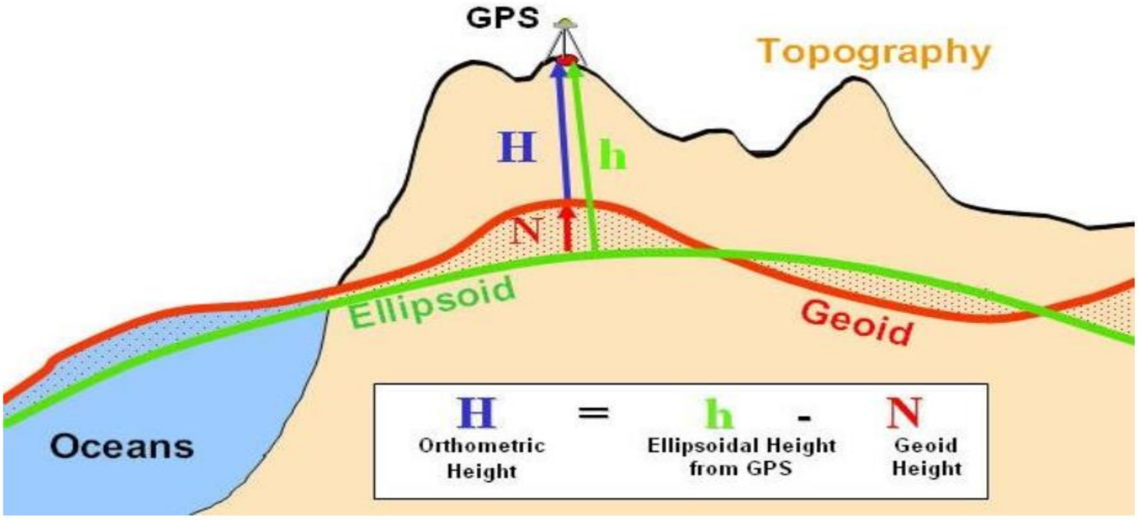

Figure 2.1: Height System (http:///www.geomobileinnovations.com/).

The geoid separation/undulation in Ethiopia is negative with the range of approximately -30m to 0m because the Orthometric height is higher than the ellipsoidal height in the region; in other words, the ellipsoid is above the surface of the geoid.

2.5 Earth Gravity Model

Earth gravitational model is a geopotential model of Earth consisting of spherical harmonic coefficients. Earth gravitational model (EGM) is used as a geoid reference of the World Geodetic System (WGS84). A vertical reference system can be defined by directly computing the gravitational potential on or above the Earth's surface from observations of gravity (NGA, 2021). Earth gravity model is a spherical harmonic model of the Earth's gravitational potential and developed through time, like EGM96 and EGM2008. There are many versions of EGMs, through time different versions were developed by different organizations these are EGM84 with degree (n) of orders of harmonic coefficients (m) equal to 180, EGM96 with degree (n) of orders of harmonic coefficients (m) equal to 360, EGM2008 with EGM84 with degree (n) of orders of harmonic coefficients (m) equal to 2160 and now EGM2008 expanded to the degree of 2190 (n=2190) and order 2159 and more precise from others. EGM2020 is a new release with the same structure as EGM2008, but with improved accuracy by incorporating newer data. For example, in 1996 NIMA (the national imagery and mapping agency) collaboration with NASA Goddard space flight center (GSFC), and Ohio state university developed and released EGM96 whereas EGM2008 was released in April 2008 by National Geospatial-Intelligence Agency. EGM96 is one global geoid model that can be used to calculate the orthometric elevation when the values of ellipsoid heights are given by GPS positioning instruments (Ibrham, 2018). The model was used to compute geoid undulations accurately to better than one meter (except for areas lacking accurate surface gravity data) and realize WGS84 as a true three-dimensional reference system. The values of the surface show at every location how much MSL varies from the ellipsoid used as a reference for GPS elevation reading (Witold franczek, 2003). Pavlis et al., (2012) defined EGM2008 as a spherical harmonic model of the Earth's gravitational potential. Over EGM96, EGM2008 represents an improvement by a factor of six in resolution, and by factors of three to six in accuracy, depending on gravitational quantity and geographic area. EGM2008 represents a milestone and a new paradigm in global gravity field modeling, by demonstrating for the first time, that given accurate and detailed gravimetric data, a single global model may satisfy the requirements of a very wide range of applications Pavlis et al., (2012). As a country, Ethiopia's EGM96 was used for geospatial data processing and analysis till 2017 (Ibrham, 2018). Also for ground data collected with differential GPS were processed by considering EGM96 again Ethiopian geospatial institute used the same model to process airborne data therefore the output DEM is having EGM96. The ground differential GPS measures ellipsoidal height (h) and is converted to orthometric height (H) using the EGM96 global geoid model. Before performing statistical analysis between the elevation model and ground truth data, GNSS-based ellipsoidal heights were transformed to orthometric heights defined by referencing the EGM96 geoid model.

2.6 Aerial Photogrammetry and Standards in Ethiopia

At the international level aerial photography (analog or film-based remote sensing) has been used as an effective mapping tool only since the early 1900s. Digital image scanners and cameras on satellites and aircraft have an even shorter history beginning in the mid-1970s. Aerial photogrammetry, which utilizes images taken from aerial or satellite platforms, followed soon after the first photographs were taken from aircraft.

In 1937, the American Society for Photogrammetry and Remote Sensing, or ASPRS established a committee to draft spatial accuracy standards. Soon after, the U.S. Bureau of the Budget published the United States National Map Accuracy Standards (NMAS) in 1941 (Congalton & Green, 2009). In the 1960s National Geospatial-Intelligence Agency (NGA), laid the statistical foundation for estimating the distribution of positional map error from a sample of reference points. In the late 1970s, the American Society for Photogrammetry and Remote Sensing's Specifications and Standards Committee started a review of the 1947 standards to update them to include standards for both hardcopy and digital maps. Different types of standards are there that are common to assess the positions of geospatial data. The most widely used type of standard is National Standard for Spatial Data Accuracy (NSSDA), United States National Map Accuracy Standards (NMAS), Principles of Error Theory and Cartographic Applications, ASPRS Interim Accuracy Standards for Large-Scale Maps, Federal Emergency Management Agency's (FEMA) Guidelines and Specifications for Flood Hazard Mapping Partners, Vertical Accuracy Reporting for Lidar Data, and National Digital Elevation Program (NDEP) Guidelines for Digital Elevation Data.

Developing and administering national spatial data infrastructure, and via spatial data collection technologies i.e. Collect, analyze, storing, and disseminating any kind of geospatial data in Ethiopia is one of the duties and responsibilities of the Information Network Security Agency (INSA, 2015) now named as Ethiopian geospatial information institute (EGII). Therefore to take over those responsibilities and duties there shall be a standard for applying, monitoring, controlling, and evaluating the country's spatial data infrastructure. Currently, EGII is equipped with the state of the art digital aerial photography acquisition and processing technologies. Although there were procedures and documents for digital aerial photography acquisition and pre-processing, this standard is more familiar with international experiences, and necessary to guide and facilitating all works related to aerial photography collection to improve the EGII's capacity in one step forward. The Standard for the Acquisition of Digital Aerial photography serves as a collective of all the specifications required for the acquisition of digital aerial photography. These standards shall be used in the preparation, delivery, and supervision of all products defined within. The effective date of the specifications and procedures within this standard is May 15, 2015. This standard shall be updated continuously, with revisions issued periodically. Some modifications to this standard are the direct result of changes in specifications and other requirements or as a result of recent experiences and technological advances. MoUHDC (ministry of urban housing, development, and construction) in Ethiopia, is also one of the mandate groups in developing the standards related to urban land cadaster.

2.7 The Role of Ground Control Points in Positional Accuracy Assessment

The NSSDA implements a statistical and testing methodology for estimating the positional accuracy of points on maps and in digital geospatial data. To geo-reference ground positions, NSSDA recommends accuracy testing by an independent source of higher accuracy is the preferred test for positional accuracy (NSSDA, 1998). The independent source of higher accuracy shall be of the highest accuracy feasible and practicable to evaluate the accuracy of the dataset.

Since the ground coordinate point is considered to be a true and more accurate coordinate than from the airborne sensor, it was used as a reference coordinate to assess the position of features collected on the airborne sensor. The ground position of checkpoints can be measured to a high degree of accuracy concerning the geodetic datum. To increase the degree of accuracy of checkpoints the DGPS reading should be taken a long session of twelve hours, forty-eight hours, and seventy-two hours. Since ground control point data is measured for a long period and simultaneously tracking many satellites locally as well as globally and processed by linking different ground satellite stations. So ground control point is considered as accurate data and can be used as references for RTK collecting data.

CHAPTER 3 MATERIAL AND METHODS

3.1 Description of the Study Area

Dire Dawa is one of two administrative cities in Ethiopia. The city is located in the Eastern part of the country on the border of Awash plain and the Eastern highland just at the foot slope of the Dengego mountain between 9°27' and 9°49'N latitude and 41°38' and 42°19'E longitude. The city is surrounded by Oromia national regional states to the Western, Northern, and Southern sides, while the Somali national regional state is bounded in the East. The city is located in the Awash River basin and the elevation varies from 950 to 2,260 meters above mean sea level. The south and south-eastern part of the city is characterized by a chain of mountains and upland at the foot of the mountain chain covering 45%; and low-lying flat land accounting for 40% of the land area (Mulugeta, 2018; Ephrem, 2006). .

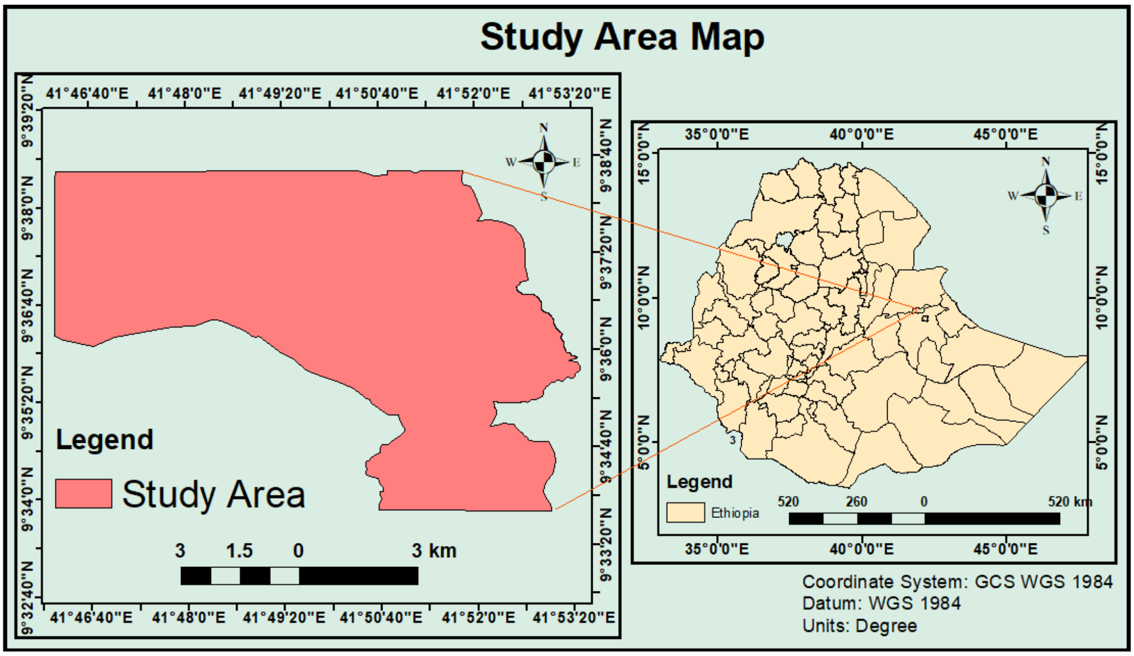

Figure 3.1: Study Area Map

3.2 Data Sources

3.2.1 Orthophoto Map

Aerial images were conducted by Information Network Security Agency (INSA) in March 2012. A high-quality charge-coupled device (CCD) ICAROS IDM 200 medium format camera with a pixel size of 6 microns having a focal length of 55.3 was used. Aerial photos acquired were end lap of 70% and side lap of 40% at a nominal flight height of 2845 m as shown in Meta data Table (Appendix H).

A georeferenced orthophoto of 15-centimeter ground sample resolution with the scale of 1:2,000 was obtained from Dire Dawa city land management bureau to assess the horizontal or planimetric accuracy. The data were acquired during the first country-based plan for cities and performed through the digital photogrammetric flight by the geospatial division of the information network security agency (INSA) now named geospatial information institute, through the airborne system of acquisition and post-processing of digital images, in the years 2012.

3.2.2 Photogrammetric DEM Data

This study has used Digital Elevation Models acquired from aerial images with requirements as listed in Meta data Table (Appendix H). The DEM is used where the end product is computed from aerial images using rigorous photogrammetric data reduction steps. Aerial images acquired were processed on “Inpho DTMaster Version 7.0 software” by the expert team. The DEM data were obtained after the following steps. The first step is considering camera calibration parameters, flight information, and aerial images to adjust interior orientation (IO). The second step determines Exterior Orientation (EO) parameters from the onboard GPS-based camera position coordinates; Inertial Measurement Unit (IMU) derived from camera attitude (yaw, pitch, and roll) and GCPs. The third step employs a rigorous least-squares adjustment technique to compute EO parameters Birhane et al., (2020).

DEM having 15-meter ground sample resolution was obtained from mandate organization of Ethiopian geospatial information institute. The vertical datum is used as a reference as EGM96. The aerial photogrammetric survey was carried out over the whole area of Dire Dawa city mainly to cities' plan of urbanization, in 2011/12 years. Figure 3.2 shows the photogrammetric DEM of the study area.

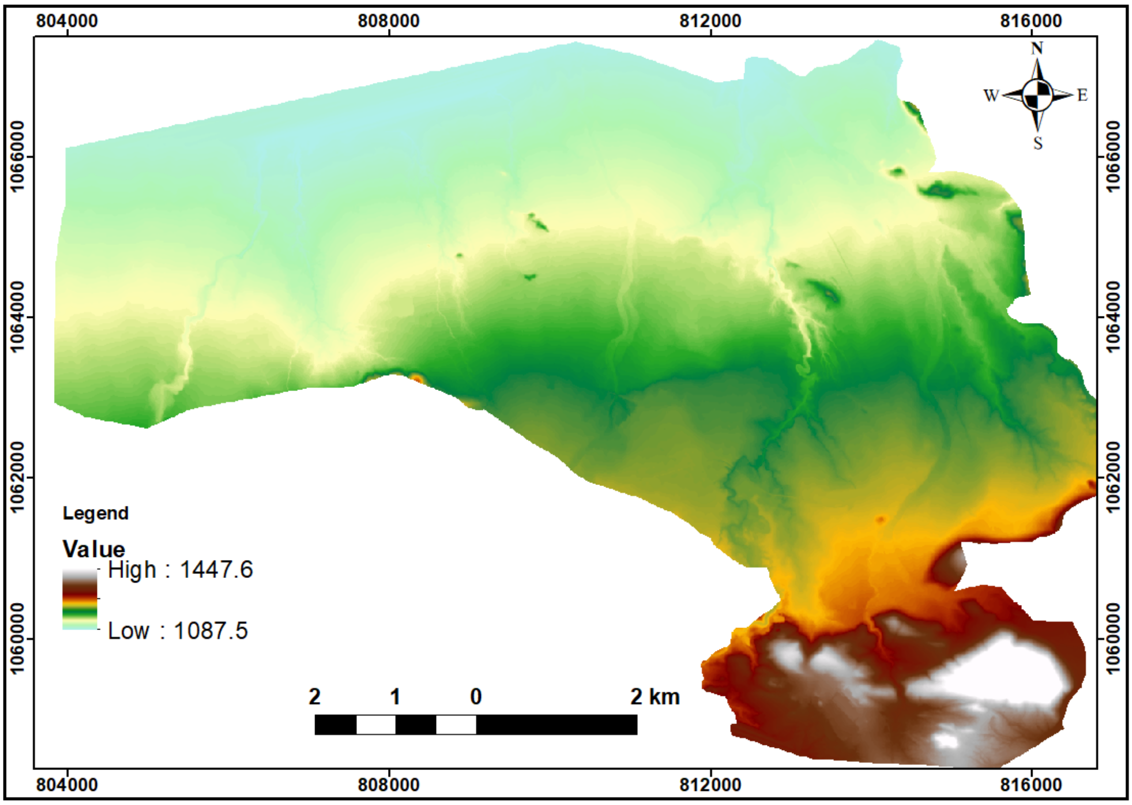

Figure 3.2: Photogrammetric DEM

3.2.3 ALOS DEM (Advanced land observing satellite) Data

ALOS DEM is also called (AW3D3-30) ALOS worldwide 3D having 30m spatial resolution. In 2015, the horizontal resolution having 30m and 3.28m RMSE vertical resolution ALOS DEM was released free of charge by the japan aerospace exploration agency (JAXA) (Takaku et.al., (2016). It gives the elevation data in detail and expresses undulations of terrain all over the world. In the study, ALOS DEM was used to show the terrain characteristics of the study area and to cross-check elevation variation or difference on it. Figure 3.4 below is the ALOS DEM of the study area.

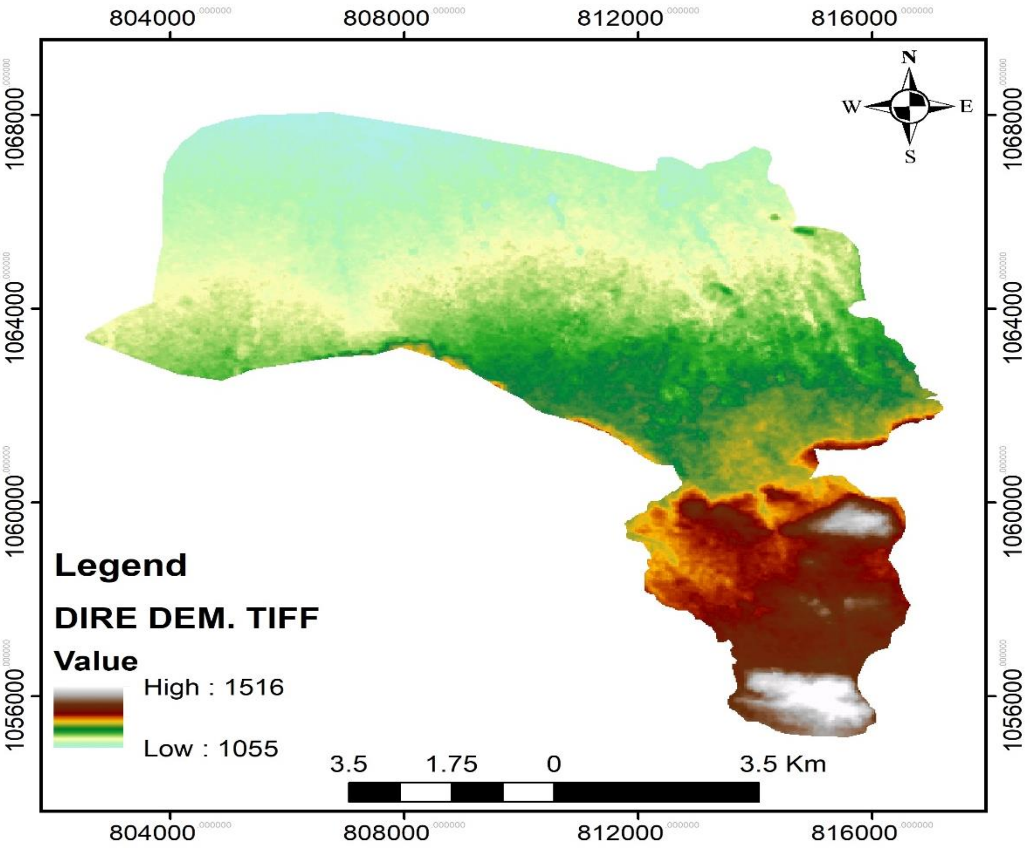

Figure 3.3: ALOS DEM

3.2.4 **Ground Control Point Data**

The reference point's accuracy would be at least three times more accurate than the DEM elevations (Athmania & Achour, 2014). For this study, Ground control points (GCP) were used as the reference point to validate the accuracy of the digital elevation model (DEM). Experts of the former Ethiopian Mapping Agency (EMA) now geospatial information institute(EGII) have collected GNSS data during the establishment of ground control points (GCP) for cadastral mapping purposes in the Dire Dawa region. During the field survey, 70 GCP points were selected and detailed measurements using Leica GNSS receivers were obtained. All measured ellipsoidal height (h) were converted to orthometric height (H) using the EGM96 global geoid model (Ibrham, 2018).

All these converted orthometric heights are used to evaluate the vertical accuracy of the DEM. The accuracy of Ground control points observed was less than 10cm in elevation.

Appendix G shows orthometric heights that are defined regarding the EGM96 geoid model and horizontally referenced with WGS1984 UTM zone 37 Adindan.

3.2.5 GPS Data

To evaluate the horizontal accuracy of the data, independent data having a better accuracy is needed, usually having three times higher accuracy. The well-defined check points were collected across the study area to evaluate the horizontal accuracy of the orthophoto. The checkpoints should be connected to the zero-order geodetic network.

At least one ground control point of base stations around the project area having zero-order is needed. In the study area, a new base station is established in connection to the nearest zero-order (IGS station). The new base station was established having a nine-hour reading session with ADDIS IGS zero-order geodetic network. Establishing the ground control point (GCP) network is a prerequisite for geo-referencing raw image data. The checkpoints were established by taking GPS measurements for at least one hour session and GPS raw data was processed using LEICA geo-office version 7.0 software. LEICA geo-office version 7.0 software is ideal for processing and analyzing satellite (GNSS) survey data recorded in the field.

Table 3.1: Data sources

No	Dataset	Source
1	Photogrammetric DEM	Ethiopia geospatial information institution
2	Orthophoto Map	Dire Dawa land administration office
3	ALOS DEM	Japan aerospace exploration agency website
4	GCP data	Dire Dawa land administration office
5	GPS data	Field Survey
6	Administrative boundary	Dire Dawa land administration

3.3 Methods

3.3.1 Checkpoint Selection

A well-defined point represents a feature for which the horizontal position is known to have a high degree of accuracy and position to the geodetic datum. For accuracy testing, well-defined points must be easily visible or recoverable on the ground, on the independent source of higher accuracy, and on the product itself. The accuracy of the datasets is tested by comparing the coordinates of several points within the data set to the coordinates of the same points acquired from an independent measurement. Points used for this comparison must be well-defined. They must be easily visible, recoverable, and measurable on the ground. The independent and test data sets must have common points (FGDC, 1999, p. 4). Since the area of interest to be tested is not rectangular it is difficult to select uniformly, an ideal distribution of checkpoints. Detailed reconnaissance surveys were occurred before setup and measuring taken on each checkpoint. Accordingly, almost well-defined and evenly distributed seven checkpoints for horizontally accuracy assessment were selected. And for vertical accuracy assessment, 70 GCP obtained were considered as a checkpoint in the study.

3.3.2 **Static GPS Survey**

Data acquisition for the independent source of higher accuracy was done using the technique of static GPS. The independent source of higher accuracy shall be of the highest accuracy feasible and practicable to evaluate the accuracy. Trimble 5800 instruments with accessories were used to take a detailed measurement in the field. The CORS international GNSS service (IGS) station in Addis Ababa and one-second order ground control points which were established after carried out nine-hour static measurement were used as a reference station. Even if the zero-order geodetic network is unavailable in the area, the zero-order station of Addis Ababa is connected to the second-order in these ways. The detailed ground measurements were observed in selected seven checkpoints and observed static GPS data processed with licensed Leica geo-office software and with online processing tools. The reading sessions of checkpoints were at least two hours 30 minutes and above except one point. Obtained GPS data in the field were downloaded in Trimble business center downloader and converted to Receiver Independent Exchange Format (RINEX) file whereas, IGS station RINEX file was downloaded through the internet linked with (http://igs.bkg.bund.de/file/rinexsearch). Lastly, the processed check point coordinate obtained were used to evaluate the accuracy of orthophoto. Figure 3.3 shows the static GPS survey of the sample checkpoint station.

Figure 3.4: Checkpoint station of static GPS survey

3.3.3 RTK GPS survey

In this study, fifty-two coordinate values were collected along the road centerlines and corners of buildings (parcels) through RTK GPS survey techniques. Out of these coordinates, 28 coordinates from parcel corners (7 parcels that is four coordinates for each parcel) and 24 coordinates from road center lines were observed. This RTK GPS surveying data collection was carried out by using Trimble 5800 instrument. During the collections period, at least four satellites were observed having a PDOP of 3, and also the reading session is from 30 seconds to 60 seconds depending on the number of satellites to be sensed. The base receiver remains stationary over the known point woreda 02 stations that exist on the top of the woreda administration building and is attached to a radio transmitter. The base station was almost the center for all rover stations. The time reading for a base receiver was for nine hours that is from 2:30 to 11: 30 local time. The base receiver started measurement two hours before the rover receiver starts measurement and stopped measuring after the rover receiver stops. The base receiver sends correction through radio and displays towards on the rover coordinates right in the field. Therefore, no post-processing was required. Then whole collected data stored in the GPS controller were exported to a USB flash disk easy as excel sheet format. The RTK GPS data are used to evaluate the accuracy of linear features such as roads centers and parcels.

3.3.4 Horizontal Accuracy Assessment

David (2015) explains that horizontal accuracy is to be assessed using root-mean-square-error (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 equations below.

$$RMSE_x = \sqrt{\frac{(\mathbf{x\ check,i - x\ test,i})^2}{n}} \qquad 3.1$$

$$RMSE_y = \sqrt{\frac{(\mathbf{y\ check,i - y\ test,i})^2}{n}} \qquad 3.2$$

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

$$RMSE_r = \sqrt{\frac{(\mathbf{x\ data,i - x\ check,i})^2 + (\mathbf{y\ data,i - y\ check,i})^2}{n}} \qquad 3.3$$

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. The NSSDA accuracy statistic worksheet is calculated by first filling out the information requested in the appropriate Table and then computing three values the sum, the average, and the root mean square error. The sum of the set of squared differences between the test data set coordinate values and the independent data set coordinate values, the average of the sum by dividing the sum by the number of test points being evaluated, and the root mean square error statistic, which is simply the square root of the average. The NSSDA statistic is determined by multiplying the RMSE by a value that represents the standard error of the mean at the 95 percent confidence level is 1.7308 when calculating horizontal accuracy.

3.3.5 **Vertical Accuracy Assessment**

The common techniques for quality assessment were based on a statistical comparison of the small reference higher quality data with the DEM to find outliers (Tomaz Podobnikar, 2009). Generally, there are two methods used to assess the vertical accuracy of spatial datasets, which are by using a statistical method and a visual method. Possible methods in the study used to assess the DEM dataset were reference test data, which is checkpoint (point data) collected in detail, processed, and considered to be three times accurate compared with DEM data generated from orthophoto of 15cm GSD product at the scale of 1:2,000. RMSE is used as a measure of vertical accuracy.

The positional accuracy statistic can be computed using the appropriate accuracy statistic worksheet. The NSSDA statistic is calculated by first filling out the information requested in the appropriate Table and then computing three values the sum, the average, and the root mean square error. The sum of the set of squared differences between the test data set coordinate values and the independent data set coordinate values, the average of the sum by dividing the sum by the number of test points being evaluated, and the root mean square error statistic, which is simply the square root of the average.

$$RMSE_z = \sqrt{\frac{(\mathbf{Z\ check,i - Z\ test,i})^2}{\boldsymbol{n}}} \qquad 3.4$$

Where:

z check i is the vertical coordinate of the i^{th} checkpoint(GCP) in the independent source of higher accuracy, z test i is the vertical coordinate of the i^{th} test dataset(DEM), n is the number of points being checked, i is an integer from 1 to n.

After computing RMSEz, the NSSDA statistic was determined by multiplying the RMSE by a value that represents the standard error of the mean at the 95 percent confidence level that is 1.9600 when calculating vertical accuracy. Notice that systematic errors have been eliminated as best as possible and vertical errors are normally distributed as reported by David (2015). Accordingly, the vertical accuracy at NSSDA 95% confidence level shall be computed by equation 3.5.

$$\text{Vertical accuracy} = 1.9600 * RMSEz \qquad 3.5$$

3.4 Data Processing and software

Nowadays many software are there for processing GNSS raw data. For this study, Leica Geo-office 7.0 software is used. First, raw GNSS data has been converted to RINEX file format because the data collected with Trimble 5800 GNSS receiver is not compatible with Leica GNSS receivers. RINEX file is receiver independent exchange format and is the standard format that all allows different types of GNSS receivers to be used together and also their data to be processed together, in all GNSS software. Until recent years, it was necessary to obtain positioning with GPS using at least two receivers, and the collected data should be processed for highly accurate positioning by using the GNSS data processing software whether scientific or commercial. However, the usage of such software is also quite difficult because they generally require deep knowledge of the GNSS and experience in processing. Furthermore, they mostly need a licensing fee this is why the online post-processing technique used and their coordinate variation compared (Reha et al., (2016).

In this study, the software is selected based on the ability to work on the existing problems in accomplishing the predetermined objectives. From the selected point the ground measurement was taken by Trimble 5800 GPS instrument and the data measured were easily exported and processed using licensed Leica geo-office software. The GPS data is also processed using online processing services to ensure the reliability of the solutions. The online post-processing technique is a web-based GNSS data processing service used as an alternative to software processing techniques. In this study, commonly used web-based online GNSS processing services are compared to each other concerning their accuracy. The popular GPS post-processing free services available are AUSPOS, Trimble RTX, OPUS, CSRS-PPP, and APPS. Trimble RTX, AUSPOS, and OPUS online services use a different technique that calculates the coordinates with a relative solution method. That means services use the fixed station points which relate to IGS and/or CORS Networks as reference points and calculates the coordinates of the points with the relative method. The next two are CSRS-PPP and APPS with the PPP technique. PPP-based services used the GNSS data collected with only a single receiver with precise satellite ephemerides and clock data by taking into account corrections like carrier phase wind-up, satellite antenna phase offset, solid and ocean tides. The use of post-processing free services saves time, minimizes cost and labor by eliminating the need for a reference station and knowledge, training, and usage of the GNSS processing software. In this study, all types of online services were used and the coordinates obtained through each were compared to each other and with coordinates of Leica geo-office software to ensure the reliability of the solutions.

On other hand, ArcGIS 10.5 software was used to create a linear feature and to more analysis. Also, NSSDA's Excel sheet was used to analyze the data statistically. Besides, global mapper software was used for the conversion of coordinates from one system to another. Since the orthophoto was georeferenced with local coordinate system and software and online post-processing were processed with the globally referenced system, therefore Easting and Northing must be transformed to the local datum that is UTM, zone 37°N.

3.5 Analysis

In this study, the positional accuracy assessment method was represented by using statistical measurements. A statistical analysis has been used to assess the accuracy of both digital orthophoto and DEM. The analysis used was based on a worksheet produced in excel sheet format developed and attacked on the website of NSSDA. The 7 (seven) checkpoints collected are used as independent highly accurate data to compare the accuracy of digital orthophoto. Also, 52 (fifty-two) check points coordinate acquired through RTK GPS were compared with the corresponding coordinates extracted from orthophoto to assess the horizontal accuracy. Then after statistical parameters such as minimum, maximum, standard deviation, RMSEr, and LE95% were analyzed. The 70 (seventy) GCP coordinates were compared with the corresponding coordinates extracted from multi-value of random points on ArcGIS software were analyzed statistically for vertical accuracy assessment. Then both coordinates were exported to excel sheet format developed by NSSDA. Finally, standard deviation, RMSEz, and LE95% (NSSDA) were computed. The diagrammatic summary of the research methodology is shown below in Figure 3.4.

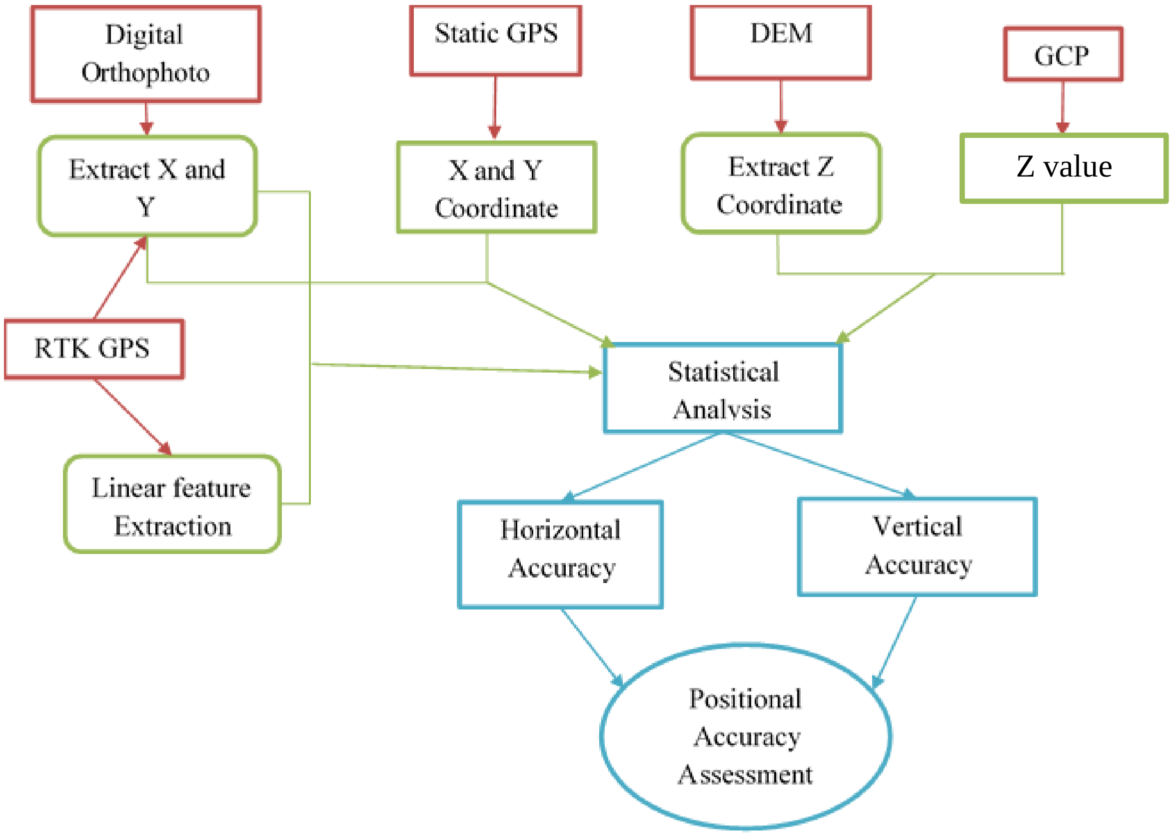

Figure 3.5: Workflow of the research methodology. Red, Input data; Green, Process; Blue, Output.

CHAPTER 4 RESULT AND DISCUSSION

4.1 Result

4.1.1 Accuracy of the Checkpoint

Both the total number and spatial distribution of checkpoints play an important role in the accurate evaluation of any geospatial data (ASPRS, 2015). As ASPRS recommendations a minimum of 20 static checkpoints for horizontal accuracy testing and 25 static checkpoints for vertical accuracy testing are evenly distributed over the area less than or equal to 500km^2. For this study, seven checkpoints were selected through the static survey technique and twenty checkpoints were selected through the RTK survey technique to validate the horizontal accuracy of orthophoto. Whereas, seventy GCP are used as checkpoints to validate the vertical accuracy of DEM. The selected checkpoints were well-defined and evenly distributed except for twenty checkpoints observed in the RTK survey technique. The accuracy of seven checkpoints observed in static survey data was less than 2cm and twenty checkpoints observed in RTK survey data were 3cm. Whereas, the accuracy of reference ground control points obtained was less than 10cm in elevation. To evaluate the positional accuracy of the data, independent data having a better accuracy is needed, usually having three times higher accuracy. Therefore, checkpoints have three times more accurate than the test datasets that we are going to validate.

4.1.2 Result of Static Survey Data

The GPS coordinates for the seven checkpoints were determined using Leica Geo-office 7.0 software having the mean RMSE error values of 0.17, 0.21, and 0.56 in Easting, Northing, and height, respectively as shown in Table 4.1 below. In addition to the mean RMSE error, the accuracy of Easting, Northing, and height are separately shown for each station. This tells that the accuracy of in-situ reference data measured is in order of sub-millimeter and millimeter level. Therefore, the measurement determined are appropriate and can be used as a reference to evaluate the accuracy of orthophoto.

Table 4.1: Result of static GPS coordinate processed on LGO.

Station Code	Easting	Northing	Height	RMSE(mm)		
				Easting	Northing	Height
Sebategna	811016.8342	1063605.185	1150.967	0.189	0.275	0.611
Maingate	811390.1334	1063945.845	1142.3581	0.101	0.123	0.405
Papa	810969.7985	1062798.223	1174.8695	0.210	0.102	0.410
Bridge	812539.4975	1062231.862	1168.7666	0.265	0.380	0.680
MA	813039.6235	1062795.05	1160.317	0.112	0.116	0.510
Mesklegna	812509.768	1063298.056	1162.8389	0.109	0.204	0.505
Triangle	811467.4437	1063439.941	1151.3459	0.214	0.290	0.790
			Sum	1.200	1.490	3.911
			Mean RMS error	0.170	0.210	0.560

GPS data acquired from the base stations (9 hours observation session) and check points (2-3 hours observation session except for station code Papa which is 1:30 hour) was processed using Leica geo-office version 7.0 software for Static survey case. The GPS data are originally processed regarding WGS 84 coordinate system and later transformed to the local coordinate of Adindan zone 37 UTM. Between two data the transformation parameters -162, -12, and +206 in Easting, Northing, and height were considered in this study, respectively. Accordingly, the coordinates that are transformed to local datum are listed in Table below.

Table 4.2: Coordinates of check points in local datum/Adindan UTM 37/.

Station Code	Easting	Northing	Orthomtric Height
Sebategna	811016.8342	1063605.185	1150.967
Main gate	811390.1334	1063945.845	1142.3581
Papa	810969.7985	1062798.223	1174.8695
Bridge	812539.4975	1062231.862	1168.7666
MA	813039.6235	1062795.05	1160.317
Mesklegna	812509.768	1063298.056	1162.8389
Triangle	811467.4437	1063439.941	1151.3459

4.1.3 Result of Real-Time Kinematics (RTK) Survey Data

Fifty-two sample points are taken with the Trimble 5800 GPS instrument having a period of 30 to 60-second reading. The base receiver remains stationary over the known point woreda 02 stations that exist on the top of the woreda administration building and is attached to a radio transmitter. The selected 52 points are at the corners of buildings, corners of concrete fences, curves of roads. All these points simultaneously exist on the orthophoto map of the city. The time reading for a base receiver was for nine hours that is from 2:30 to 11: 30 local time. The expected positioning accuracy is of the order of 2 to 5 cm (RMS) on the field was 3cm in these cases. The base receiver started measurement two hours before the rover receiver starts measurement and stopped measuring after the rover receiver stops. The base receiver sends correction through radio and displays towards on the rover coordinates right in the field. Therefore, no post-processing was required. The whole collected data stored in the GPS controller were exported to USB flash disk easy as excel sheet format as shown in Appendix B. The RTK GPS data are used for evaluating the accuracy of linear features such as roads and parcels.

4.1.4 Reliability of GPS solution acquired from different online post-processing techniques

Conventional methods of surveying use at least two dual-frequency receivers and antennas but, today geodetic survey control networks on regional and national scales can be surveyed with only one dual receiver and free processing in web-based online service. Using only one receiver and a free post-processing service will help to reduce the personnel, logistics, and equipment costs compared to the conventional approach (Sarhat Adam, 2017). Different online post-processing tools are available today to process GPS data. The RINEX file was uploaded to the web-based online global navigation satellite system (GNSS) processing services through e-mail or the service’s web page. The RINEX file is usually processed in static mode by calculating the coordinates with a relative solution approach (e.g. Trimble center point real time extended post-processing (Trimble RTX), Australian positioning service (AUSPOS); or by calculating the coordinates with a precise point positioning (PPP) solution approach technique (e.g. the Canadian spatial reference system (CSRS-PPP), Automatic Precise Positioning Service (APPS). The online processing tools used in the study were to compare the accuracy level of the GPS solutions of Easting and Northing with the Leica geo-office version 7.0 GPS solutions of Easting and Northing. Table 4.3 shows GPS solutions of the seven checkpoints derived from CSR-PPP, TRIMBLE, APPS, and AUSPOS online processing services.

Table 4.3; Easting and Northing of seven checkpoints obtained by different online processing tools.

Station Code	CSRS-PPP		TRIMBLE		APPS		OPUS		AUSPOS	
	Easting	Northing	Easting	Northing	Easting	Northing	Easting	Northing	Easting	Northing
Sebategna	811016.800	1063605.744	811016.497	1063605.319	811389.723	1063946.002	811017.186	1063605.057	811016.443	1063605.330
Main gate	811389.857	1063946.267	811389.432	1063945.802	810969.394	1062798.381	811390.532	1063945.696	811389.761	1063946.000
Papa	810969.986	1062798.561	810969.242	1062798.162	812539.794	1062232.044	N/A	N/A	N/A	N/A
Bridge	812539.074	1062232.487	812539.009	1062231.810	813039.223	1062795.198	810969.049	1062798.398	812539.569	1062231.750
MA	813038.896	1062795.287	813038.876	1062795.017	812509.356	1063298.201	813039.413	1062795.156	813039.214	1062795.200
Mesklegna	812509.435	1063298.112	812509.096	1063298.028	811467.022	1063440.082	812509.629	1063298.277	812509.469	1063297.735
Triangle	811467.114	1063440.449	811466.792	1063439.902	811016.537	1063605.317	811467.086	1063439.981	811467.641	1063440.067

The study also tried to see the residuals between GPS solutions of online processing tools one to another. The residual between CSRS derived coordinates and the corresponding coordinates obtained from TRIMBLE and APPS as well as the TRIMBLE derived coordinates with the corresponding derived coordinates obtained from APPS were compared according to the residuals. The result tells us the difference in Easting and Northing are both in centimeter and decimeter accuracy level. Table 4.4 shows the residuals between GPS solutions of different online processing tools.

Table 4.4: The residual between different online processing tools

Station	CSRS -TRI		CSRS – APPS NASA		TRIMBLE – APPS NASA	
	Easting	**Northing**	**Easting**	**Northing**	**Easting**	**Northing**
Sebategna	0.303	0.425	0.263	0.427	0.040	0.002
Main gate	0.425	0.465	0.134	0.265	0.291	0.200
Papa	0.744	0.399	0.592	0.18	0.152	0.219
Bridge	0.065	0.677	-0.72	0.443	0.785	0234
MA	0.02	0.27	-0.327	0.089	0.347	0.173
Mesklegna	0.339	0.084	0.079	-0.089	0.230	0.180
Triangle	0.322	0.547	0.577	0.789	-0.231	-0.183

The process through web-based coordinates was then compared with the reference coordinates (conventionally), i.e. coordinates processed through Leica geo-office 7.0 were used to analyze the online processing service's accuracy. The Leica geo-office (LGO) software used for processing GNSS data is one of the processing software. The reference coordinate system used was the world geographic system (WGS 84), while the web-based online services computed coordinates are provided in International Terrestrial

Reference Frame (ITRF-2014) co-ordinate system. The difference in the coordinate system has a great effect on coordinates. To make the coordinate system the same reference the web-based online services computed co-ordinate provided in International Terrestrial Reference Frame (ITRF-2014) co-ordinate system were converted to WGS 84 by global mapper software. Therefore, the analysis has been performed to compare coordinates of positions in the WGS84 coordinate reference system. The study also tried to see the residuals between the GPS solution of online processing tools on one side and the GPS solution of LGO 7.0 software on another. The difference between LGO 7.0 software-derived coordinates and the corresponding coordinates obtained from online processing tools (CSRS, TRIMBLE, APPS, AUSPOS, and online positioning user service (OPUS). In addition to residuals (RMSE) the min, max, standard deviation, and LE95% statistical parameters were analyzed. The result is shown in Table 4.5, and it shows the change in Easting and Northing of LGO 7.0 software from different online processing. The variation between the two processing reflects there is decimeter level.

Table 4.5: Residuals of LGO derived coordinates with CSRS, Trimble, APPS, AUSPOS, and OPUS.

Station Code	LGO - CSRS		LGO - TRIMBLE		LGO - APPS		LGO - AUSPOS		LGO - OPUS	
	Δx	Δy	Δx	Δy	Δx	Δy	Δx	Δx	Δx	Δy
Sebategna	0.034	-0.559	0.337	-0.134	0.297	-0.132	0.391	0.391	0.352	-0.128
Maingate	0.276	-0.422	0.701	0.043	0.410	-0.157	0.372	0.372	0.399	-0.149
Papa	-0.188	-0.338	0.557	0.061	0.405	-0.158	N/A	N/A	N/A	N/A
Bridge	0.423	-0.625	0.488	0.052	-0.297	-0.182	-0.071	-0.071	0.304	-0.175
MA	0.728	-0.237	0.748	0.033	0.401	-0.148	0.409	0.409	0.211	-0.105
Mesklg	0.333	-0.056	0.672	0.028	0.412	-0.145	0.299	0.321	0.139	-0.221
Triangle	0.330	-0.508	0.652	0.039	0.422	-0.141	-0.198	-0.216	0.358	-0.127
MIN	-0.188	-0.625	0.337	-0.134	-0.297	-0.182	-0.072	-0.072	0.352	-0.159
MAX	0.728	-0.056	0.748	0.061	0.422	-0.132	0.410	0.410	0.399	-0.127
STDV	0.290	0.198	0.143	0.068	0.263	0.016	0.232	0.131	0.024	0.016
RMSE	0.539	0.445	0.378	0.260	0.513	0.126	0.481	0.362	0.155	0.125
LE95%	0.932	0.771	0.655	0.451	0.889	0.219	0.833	0.626	0.268	0.216

When comparing the RMSEr and NSSDA horizontal accuracy at 95% confidence level of orthophoto with LGO and OPUS, OPUS results in 40cm and 0.63cm whereas, LGO results in 0.22cm and 39cm, respectively. This means that LGO yields better accuracy than OPUS services. It can be seen from Table 4.5, is that the expected standard deviation of OPUS was the smallest difference it provides more accurate geodetic solutions from other online processing tools and is more appropriate with LGO 7.0 version software whereas, CSRS-PPP were the largest differences for both the x and y coordinates and provides the less accurate geodetic solution. According to RMSE error result OPUS, TRIMBLE, AUSPOS, and CSPS are ranked as first, second, third, and fourth in accuracy rank level, respectively. The APPS results well in RMSE error next to OPUS in y-direction but in the fourth rank in the x-direction. The minimum, maximum, and root mean squared values that are associated with the online service solutions are shown in the same table above, and specifically root mean squared error is presented in a more illustrative way using histogram Figure. Figure 4.3 explains RMSE error in the x-direction and Figure 4.4 explains RMSE error in the y-direction.

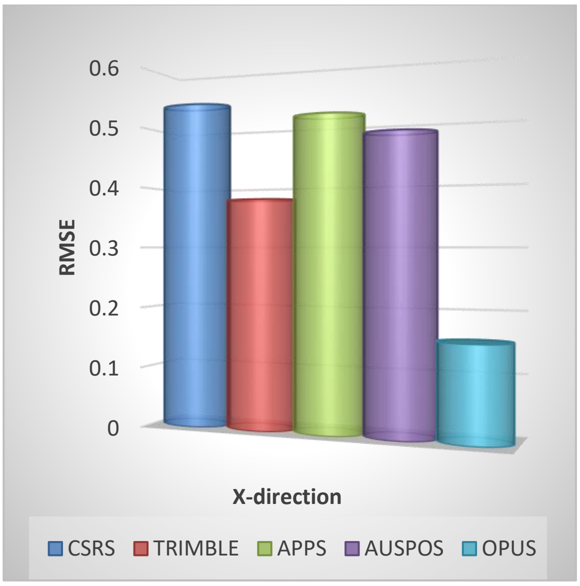

Figure 4.1: Plot of residual errors in the x-direction

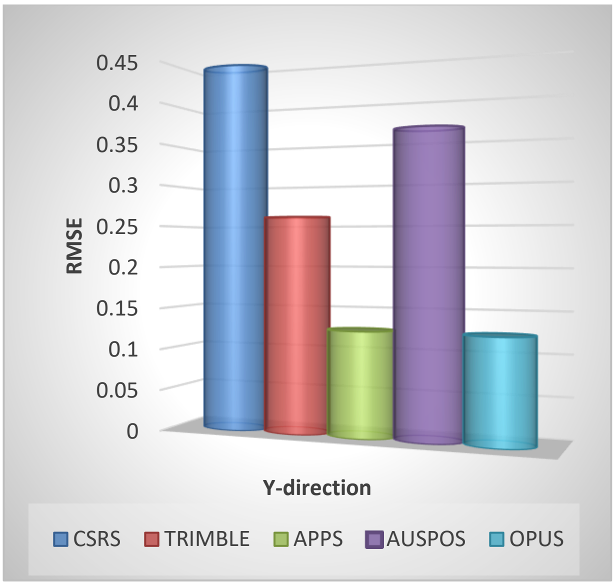

Figure 4.2: Plot of residual errors in the y-direction

4.1.5 **Horizontal Accuracy Assessment Result**

4.1.5.1 Assessment Based On Static Survey Data

This study specifically assesses the horizontal accuracy of orthophoto in two methods. The first method is when the accuracy of orthophoto is assessed with the reference to the static GPS survey result, and the second is by assessing the orthophoto with the reference to the RTK GPS survey result. A point-based assessment technique was used to assess orthophoto accuracy. To do this seven independent checkpoints coordinates were obtained and each checkpoint was independently compared with the coordinates obtained from orthophoto. The NSSDA verifies that independent data set having higher accuracy that corresponds to the data set being tested should be selected to assess the accuracy level of spatial data. According to Geospatial Positioning Accuracy Standards, the independent source of higher accuracy shall be the highest accuracy feasible and practicable to evaluate the accuracy of the dataset. The independent data set for these cases was ground GPS data collected in conventional field surveys for research cases because GPS data is considered to be accurate. Horizontal accuracy shall be tested by comparing the x and y coordinates of well-defined points in the dataset with coordinates of the same points obtained from an independent source of higher accuracy. For these cases, the x, y coordinates of highly accurate independent points (ground survey data) were compared with the x, y coordinates of tested (digitized orthophoto coordinates) of the same points. To estimate the accuracy of spatial data the national standards for spatial data accuracy uses the root-mean-square error (RMSE) statistically. The accuracy (RMSE) of the tested data set or orthophoto, as compared to the GPS data or independent data acquired from LGO processing, is approximately 12.90 cm (0.13m) and 18.4cm (0.184m) in x and y-direction, respectively as shown in Table 4.6 below. The overall RMSE of orthophoto was having 22.5 cm. In other words, the positional accuracy of orthophoto evaluated for seven checkpoints is estimated as 39cm at a 95 % confidence level. The NSSDA 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 (NSSDA, 1998). From the above result the RMSE of orthophoto coordinates and in-situ static GPS coordinates of similar checkpoints in x and y-directions and 95% confidence level.

Table 4.6: Residual in Easting and Northing coordinates obtained from orthophoto and the GPS through the static survey.

Station Code	**Coordinates derived from GPS**		**Coordinate derived from orthophoto**		**Residuals(Errors)**	
	Easting	**Northing**	**Easting**	**Northing**	**Δ Easting**	**ΔNorthing**
Sebategn	811016.834	1063605.185	811016.59	1063605.422	0.244	-0.237
Maingate	811390.133	1063945.845	811390.027	1063945.647	0.106	0.198
Papa	810969.798	1062798.223	810969.719	1062798.311	0.079	-0.088
Bridge	812539.497	1062231.862	812539.37	1062231.657	0.127	0.205
MA	813039.623	1062795.05	813039.487	1062794.761	0.136	0.288
Mesklgna	812509.768	1063298.056	812509.822	1063298.11	-0.054	-0.054
Triangle	811467.444	1063439.941	811467.414	1063439.866	0.0297	0.075
				MIN	0.0297	0.054

MAX		0.244	0.289
Standard deviation		0.020	0.030
RMSE		0.129	0.184
RMSEr	**0.225**	SQRT($RMSEx^2$ +$RMSEy^2$)	
NSSDA at 95% confidence level	**0.389**	RMSEr*1.7308	

The obtained RMSE and LE95% meet the ASPRS 2013 standard and it can be applied for the production of large-scale planimetric mapping. Similarly, orthophoto meets 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.

4.1.5.2 **Assessment Based On RTK Survey Data**

Coordinate values obtained using RTK GPS were compared with the corresponding coordinates extracted from orthophoto. To do this different linear features (such as road centerlines and parcels) were considered to assess the horizontal accuracy of orthophoto. In this specific study, the accuracy of orthophoto is assessed with the reference to the data based on RTK GPS survey results. To achieve the objective, line and polygon features are selected for the study around the main campus of Dire Dawa University. Road center from two directions (Sebategna and main campus) to Tooni having around 500m ground length road line and 7 parcels are being selected. In this study, 52 coordinate values were collected along the road centerlines, corners of buildings, and sharp corners of fences. The coordinates of the linear features were obtained in real-time kinematic global positioning system mode of techniques (RTK GPS). Since the mode is real-time that is the error during measurement is corrected or processed on the field directly in the way of correction sent from the base station to rover stations through a radio link. The obtained coordinate values through RTK GPS were imported to the ArcGIS software environment and linear features were created, digitized, and compared with extracted linear features from the orthophoto map.

On another side, linear features are extracted from the orthophoto map easily by directly digitizing features from the orthophoto sheet according to selected parcel corners and road centerlines at the same point on the ground with RTK data collected. Generally, when we compare linear features (road centerline and parcel) accuracy, it has good accuracy and it lies on an acceptable standard that is demanded the production of large scale maps. Comparative maps between digitized orthophoto parcel (red color) and RTK survey point of sample parcel feature (black color) are shown in Figure 4.3. The map below does not shows the statistical assessment, rather it shows the variation in the orientation of parcel features. To statistically assess the accuracy of parcel features five (5) parcel corners total of twenty checkpoints were used in this study. The coordinates and statistical analysis is done through the RTK survey were listed in Table 4.7 (Pc to represent parcel corner).

Table 4.7: Residual in Easting and Northing coordinates obtained from orthophoto and the GPS through RTK survey.

Station Code	**Coordinates derived from GPS**		**Coordinate derived from orthophoto**		**Residuals(Errors)**	
	Easting	**Northing**	**Easting**	**Northing**	**∆ Easting**	**∆Northing**
Pc1	811638.902	1063021.31	811638.93	1063021.15	-0.023	0.161
Pc2	811303.52	1063139.41	811304.26	1063139.07	-0.738	0.343
Pc3	811367.234	1063324.28	811367.42	1063323.96	-0.186	0.318
Pc4	811433.576	1063302.13	811433.75	1063301.88	-0.172	0.25
Pc5	811436.279	1063314.09	811437.07	1063313.92	-0.787	0.17
Pc6	811504.959	1063291.94	811504.98	1063291.84	-0.025	0.102
Pc7	811554.167	1063274.81	811554.99	1063274.58	-0.827	0.232
Pc8	811658.072	1063240.07	811658.27	1063239.94	-0.196	0.137
Pc9	811671.06	1063278.12	811671.17	1063277.81	-0.106	0.311
Pc10	811671.942	1063266.71	811671.97	1063266.5	-0.029	0.208
Pc11	811664.158	1063238.14	811664.55	1063237.82	-0.394	0.317
Pc12	811707.076	1063212.15	811707.55	1063211.85	-0.476	0.301
Pc13	811567.465	1063311.07	811567.63	1063310.83	-0.169	0.237
Pc14	811543.074	1063408.18	811543.63	1063408.00	-0.56	0.182
Pc15	811474.995	1063432.3	811475.56	1063432.00	-0.565	0.306
Pc16	811466.455	1063436.71	811467.44	1063436.46	-0.982	0.245
Pc17	811459.501	1063413.17	811460.03	1063412.97	-0.527	0.194
Pc18	811403.005	1063432.01	811403.04	1063431.9	-0.032	0.119
Pc19	811410.477	1063456.17	811410.97	1063455.91	-0.497	0.259
Pc20	811720.815	1063250.3	811721.15	1063249.92	-0.338	0.379
				MIN	-0.982	0.102
				MAX	-0.023	0.379
				Standard deviation	0.246	0.0377
				RMSE	0.26	0.15
			RMSEr	0.30	SQRT($RMSEx^2$ $+RMSEy^2$)	
			NSSDA at 95% confidence level	0.52	RMSEr*1.7308	

Statistically, the agreement between the two datasets can be exemplified by analyzing the mean and standard deviation of the difference between the coordinates from parcel corner of orthophoto and RTK GPS surveying for only five parcel corners from both datasets. The mean of the difference is -23.05cm and 6.3cm in x- and y-components, respectively. Whereas, the standard deviation of the difference is 24.6cm and 3.77cm in x- and y- components, respectively. The overall RMSE of orthophoto was having 30cm. In other words, the positional accuracy of orthophoto evaluated for twenty checkpoints is estimated as 52cm at a 95 % confidence level through the RTK survey.

The result of standard deviation (i.e. 24.9cm) shows a good measure of the accuracy of orientation in linear feature and the results achieved in this study are within the national standard of error budget as explained by Zinabu et al., (2017). .

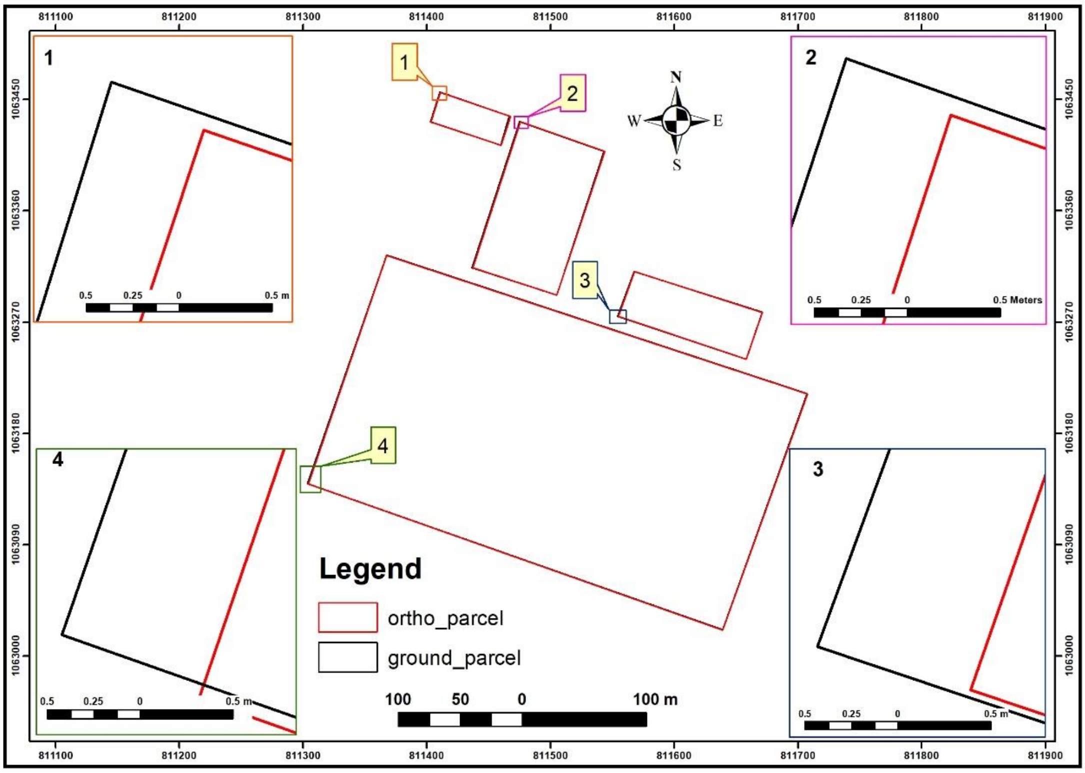

Figure 4.3: Comparative map between digitized orthophoto and RTK survey point of sample parcel feature**.**

Comparative maps between digitized orthophoto road centerline (red color) and RTK survey point of sample road centerline (blue color) are shown in Figure 4.4. The map below does not shows the statistical assessment, rather it shows the variation in the orientation of the road centerline.

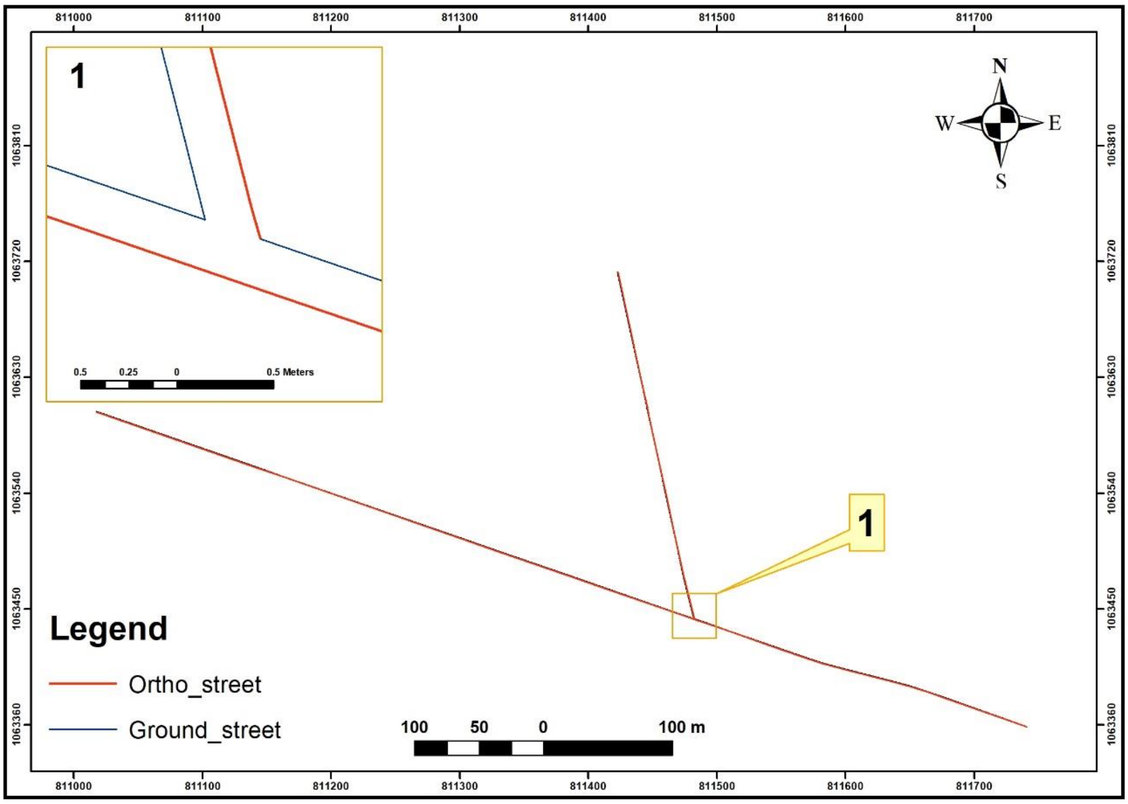

Figure 4.4: Comparative maps between digitized orthophoto and RTK survey point of sample road centerline.

4.1.6 Vertical Accuracy Assessment Result

This study has used ASPRS 2013 standards for assessing the vertical accuracy of the DEM against ground GCP data because ASPRS 2013 standard is designed to be dependent on GSD ground sample distance (GSD). The vertical accuracy is determined by multiplying ground sample distance by a common factor of 1.960. So the expected vertical accuracy of DEM to have less than or equal to 29.4cm (15cm*19.60) because the GSD of data is 15cm using the checkpoint of RMSEz and LE95%. The obtained height or elevation data with the DEM at an RMSEz of 4.06m and the vertical accuracy of the DEM is about 7.95m at a linear error of 95% confidence level. According to NSSDA's standard, the result shows that the photogrammetric DEM does not meet the vertical accuracy requirement of ASPRS 2013 standards as well as the standards of the Ethiopian geospatial information institute (EGII), i.e., RMSEz of ±0.45m for a map scale of 1:2,000 (MoUDHC, 2015). Also, it has no agreement with ASPRS 2015 standards. The statistical analysis done for vertical accuracy assessment was listed in Table that is shown in Appendix I.

Uuemaa et Al., (2020) found that slope had the strongest effect on DEM accuracy. The accuracy assessment based on slopes helps to assess the dependency of vertical accuracy on terrain characteristics. To do this elevation points as much as possible should be evenly distributed across different terrain characteristics throughout the study area. The slope of the terrain can be derived from digital elevation model data in the computer aid system of ArcMap software. The terrain slope from ALOS DEM was computed and reclassified into four classes ranging from 0-5% flat, 5-10% rolling, 10-25% Mountains, and 25 up to 50% escarpment. The elevation of the area is rugged and has a negative effect on data accuracy. The terrain can be flat, rolling, mountains, and escarpment regarding the percent of slope according to Ethiopia Road Authority (ERA) standard (ERA, 2002).

Table 4.8: The RMSE and LE95% in dependency of terrain characteristics.

Terrain characteristics	Slope class	Min	Max	Mean error	Standard deviation	RMSEz (m)
Flat	0-5	0.007	1.29	0.60	0.396	0.77
Rolling	5-10	1.34	2.91	2.04	0.491	1.43
Mountainous	10-25	2.96	8.86	4.26	1.670	2.06
Escarpment	25-50	14.6	64.9	30.9	17.51	5.56

From the results of Table 4.8, the standard deviation of elevation difference increases with increasing terrain characteristics, which is small deviation fails in flat terrain and large deviation fails in mountainous as well as escarpment terrain. Müller et al., (2014) identified slope as the main factor that determines the accuracy of the digital elevation model and it shows the dependency of RMSE on a slope. DEMs generate accurate data for slopes ranging from 0% to 5%. The accuracy of the DEMs starts to decrease in steeper slopes. Especially in the very steep slope the RMSE increases drastically (Müller et al., (2014). The elevation accuracy decreases consistently as slopes increase and they are almost linearly correlated (Toutin, 2002). According to the result acquired in column two in Table 4.8 in flat slope (0% - 5%), the vertical accuracy in flat characteristics meets ASPRS 2013 standards. And the standards of the Ethiopian geospatial information institute (EGII), i.e., a standard deviation of ±0.45m for a map scale of 1:2,000 because 0.396m is less than 0.45m. But rolling (14.6% error), mountainous (21% error) and escarpment (56.6% error) terrain characteristics result in high RMSEz and having low vertical accuracy. The reclassified slope of ALOS DEM in the background with elevation difference between the photogrammetric DEM and ground control point is represented in the map in Figure 4.5. The dependency of vertical accuracy in terrain characteristics is true according to the result obtained and the methodology used.

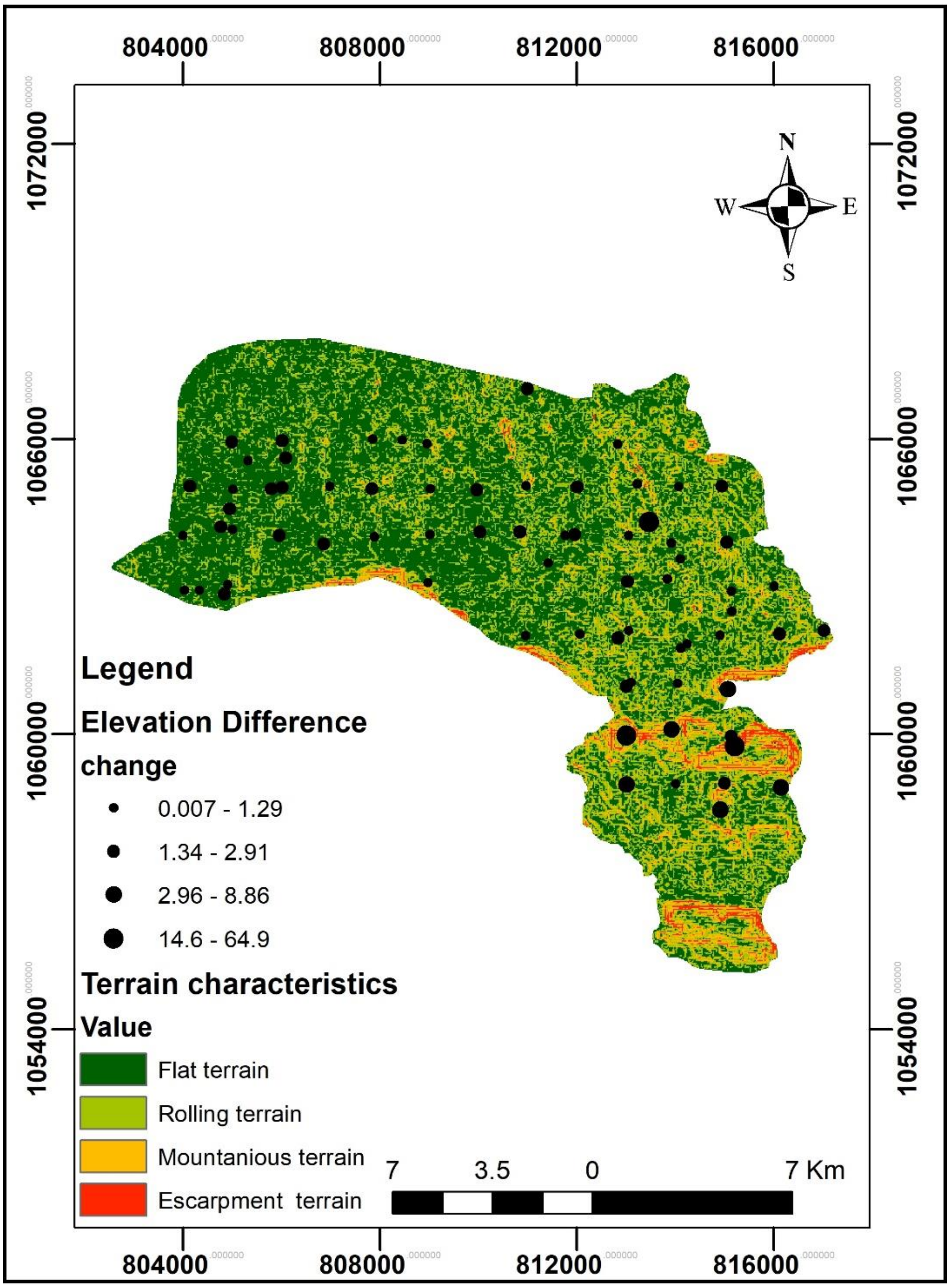

Figure 4.5: Elevation variations with respect to terrain characteristics

4.2 Discussion

Nowadays the terrestrial way of surveying and mapping almost has been changed to an easier aerial-based sensor mode of technology from space based on the scope of application. It saves time, manpower, and therefore, cost also. But, the question here is that how much accurate is it and this study target to answer the positional accuracy of both orthophoto and DEM of Dire Dawa city. Easting, Northing, and the height of aerial photogrammetry data were assessed with ground truth in the study. Digital orthophoto and DEM are some of the main aerial photogrammetric products used for many applications in different sectors. The research being done titled with accuracy assessment of digital orthophoto were not new for the study area, but accurate assessment of digital DEM were not yet in research level and published it may be in office level.

During the study time, the researcher compared digital orthophoto relative to ground survey coordinate, and the RMSE for each data set was computed. Also compares the digital orthophoto related with both RTK GPS and Static GPS surveying points to minimize RMSE because if the session of taking a long time makes the raw data becomes accurate and the RMSE should be a minimum.

In this study, the positions of both horizontal (x and y) and vertical (z) coordinates of aerially developed digital orthophoto and digital elevation model of Dire Dawa were assessed based on the criteria listed at the national and international level of standards. According to the result, the digital orthophoto generated by the mandated agency in producing the products of geospatial data in the country Ethiopia. It satisfies horizontal accuracy in both national (EGII) and the ASPRS standards. However, the vertical accuracy of the digital elevation model used in the study cannot meet both EGII and ASPRS standards of geospatial data.

Finally, I get RMSE of 12.9cm and 18.4cm in the x and y direction and horizontal accuracy of 25.5cm respectively. Whereas other got on same study area were RMSE of 13.8cm and 15cm on X and Y direction respectively in static survey technique for the production of large-scale planimetric maps (Mulugeta, 2018). And the horizontal accuracy of the parcel was 30cm having RMSE of 26cm and 15cm in x and y direction respectively in the RTK survey technique. The same study got 45cm on horizontal accuracy and RMSE of 17.3m and 19.6m on x and y directions respectively.

The RMSE of my result were 4.06m and vertical accuracy of 7.95m, in Mekelle city, the related research results standard deviation of 51cm and RMSE of 85cm (Birhane et al., (2020). During that study, the researcher verified that DEM does not meet the accuracy of EGII.

The accuracy decreases with slope increases with the standard deviation of 20.3m, and (Kyaruzi, 2005) also found 30m between low and steep slope in standard deviation.

Furthermore, the study also compares the coordinates processed by LGO software with those processed through the web-based online processing service. Accordingly, the OPUS result shows the least RMSE error comparing with others. That is the processed coordinate value of OPUS and software is almost the same, so any user can access OPUS online processing as well.

Chapter 5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Positional accuracy assessment of a digital orthophoto map and photogrammetric DEM is a very important task due to its wide range of applications. Horizontal accuracy assessment of digital orthophoto map is performed with ground independent data. It is done by comparing the ground coordinates of checkpoints with the same point from orthophoto map. And this is also true for vertical accuracy assessment of photogrammetric DEM. In this study, a total of seven checkpoints were acquired through Static survey and fifty-two checkpoints acquired through RTK survey technique selected for assessment of orthophoto map, and seventy ground control points are selected for assessment of photogrammetric DEM.

In this study, both horizontal and vertical accuracy assessment of orthophoto map and photogrammetric DEM is studied, respectively. Statistical analysis has been done to assess the accuracy of both datasets. In addition to this, a slope was derived from DEM to show the dependency of vertical accuracy in terrain characteristics. The horizontal accuracy assessment was compared in terms of RMSE and 95 % confidence level and RMSE in the x-direction and y-direction from the two datasets (RTK GPS survey checkpoints and derived coordinates of digital spatial data). The vertical accuracy assessment accuracy was evaluated mainly in terms of RMSE and 95 % confidence level in the z-direction.

According to the data being used, the methodology and having the result get the orthophoto map to satisfy the national standard of urban cadaster (that is horizontal RMSE does not exceed 0.45m). Whereas, photogrammetric DEM does not achieve the ground elevation accuracy that may be due to the undulating terrain or it may be less quality during the generating of DEM. The accuracy of the DEM dependency on terrain characteristics or slope. Accordingly, RMSE in flat terrain has less error and is more accurate and escarpment has the highest error and is less accurate. The vertical accuracy in flat terrain characteristics meets both international and national standards whereas, rolling terrain, mountainous terrain, and Escarpment terrain are not. This shows that vertical accuracy depends on terrain characteristics.

Different web-based online post-processing services (AUSPOS, ASPRS, OPUS, TRIMBLE, CSRS-PPP, APPS) were compared with LGO software post-processed data and with one another. When comparing the RMSEr and NSSDA horizontal accuracy at 95% confidence level of orthophoto with LGO and OPUS, OPUS results in a higher error and less accuracy whereas, LGO results in a low error and higher accuracy, respectively. This means that LGO yields better accuracy than OPUS services. The expected standard deviation of OPUS was the smallest difference with LGO whereas, CSRS-PPP were the largest differences for both the x and y coordinates. According to RMSE error result OPUS, TRIMBLE, AUSPOS, and CSPS-PPP are ranked as first, second, third, and fourth in accuracy rank level, respectively. The APPS results well in RMSE error next to OPUS in y-direction but in the fourth rank in the x-direction. Therefore, the first three ranks best fit the software post-processing and can be used instead of it.

5.2 Recommendation

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. It is also recommended to implement an accuracy assessment of DEM especially topography having undulating terrain. The number of checkpoints from different terrain characteristics should be approximately the same and it should be sparsely distributed throughout the area. For long baseline distance between stations and long reading sessions of data, web-based online processing services have a better result with commercial GNSS software packages like LGO. Further work will be needed with up-to-date aerial or ground data through growing technology with varying accuracy requirements.

References

Abdullah, D. Q., Maune, D. D., Smith, D., & Heidemann, H. K. (2014). *ASPRS Positional Accuracy Standards for Digital Geospatial Data.* ASPRS Map Accuracy Standards Working Group. Photogrammetric Engineering & remote sensing. Retrieved March 2015

Alkan Reha Metin, Ibrham Murat Ozulu, & İlci, V. (2016, may). Web-based GNSS data processing services as an alternative to conventional processing techniques. *ResearchGate*.

ASPRS. (1990). ASPRS accuracy standards for large-scale maps. The *American standard of photogrammetry and remote sensing.*

ASPRS. (2015, March). ASPRS Positional accuracy standards for digital geospatial data. *Photogrammetric Engineering and Remote Sensing, 18*.

Athmania, D., & Achour, H. (2014). External validation of the ASTER GDEM2, GMTED2010, and CGIAR-CSI-SRTM v4.1 free access digital elevation models (DEMs). *Remote Sensing, 6*.

Binyam . (2015). Land investment for transformation. *LIFT program.*

Birhane, H., B. B. Tulu, B., Berhanne, G., & Martin, V. (2020). Validation of airborne sensor photogrammetric digital terrain and global digital elevation models around Mekelle, Ethiopia.

Boye, C. B; Appeaning Addo, K. and Williams, E. A. (2016, June). Positional accuracy assessment for effective shoreline change. *Ghana Mining Journal*, *16*, 1.

Congalton G.R. (2001). Accuracy assessment and validation of remotely sensed and other spatial information. *ResearchGate*, 1-10.

Congalton, & Green. (2009). *Assessing the accuracy of remotely sensed data, Principles, and practices second edition.* Taylor & Francis group. Retrieved from http://www.taylorandfrancis.com

Dzigbordi, W. (2013). Accuracy checks in the production of orthophotos. *Journal of environment and earth science, Vol. 3, No.12*.

Ephrem, G. (2006). Preliminary assessment of Dire Dawa flood damage. *unpublished*.

Esri. (2020). Retrieved from https://desktop.arcgis.com/en/arcmap/latest/extensions/main/about-arcgis-for-desktop-extensions.htm.

FDRE. (2003, March). Grids & Datums. *Photogrammetric engineering and remote sensing*.

FGDC. (1984). The accuracy of geodetic control surveys is evaluated using Part 2, Standards for geodetic networks.

FGDC. (1998). Geospatial positioning accuracy standards Part 3: National standard for spatial data accuracy. Virginia: FGDC-STD-007.3.

FGDC. (1999). Positional accuracy handbook. Minnesota: Minnesota planning land management information center. Retrieved from http://www.lmic.state.mn.us/gc/gc.htm

Ghandehari, Buttenfield, & Carson. (2019). Comparing the accuracy of estimated terrain elevations across the spatial resolution. *International Journal of Remote Sensing*.

Green, C. a. (2009). Assessing the accuracy of remotely sensed data, Principles, and Practices.

Hareya B., & et.al. (2020). Validation of airborne sensor photogrammetric digital terrain and global digital elevation models around Mekelle, Ethiopia. Retrieved from https://creativecommons.org/licenses/by/4.0

Hebeler, F., & Purves, R. (2009). The influence of elevation uncertainty on the derivation of topographic indices. *Geomorphology,* 4-16.

Ibrham, A. (2018, Nov). *Vertical accuracy assessment of open source digital elevation model using GPS point and reference DEM over Ethiopia*.

ICA. (1991). *International cartographic association on Spatial database transfer standards.* London and Newyork: Elsevier science publisher LTD.

INSA. (2015). *Digital aerial photography acquisition standard.* Geospatial data acquisition and processing team. Addis Ababa: Aerial surveying sub-team.

Li, J., Chapman, M., & Sun, X. (2006). Validation of satellite-derived digital elevation models from in-track IKONOS stereo imagery.

Mark, A. B., & Tina, W. D. (2013). Accuracy Checks in the Production of Orthophotos. *Journal of Environment and Earth Science, Vol. 3*, 5-6.

Martin V.; Getachew T. A. (2018). *Digital Aerial Mapping.* martin.vermeer@aato.fi and sget2006@yahoo.com.

Maune, D. (2015). *New ASPRS positional accuracy assessment standards for digital geospatial data.* deweberry.

MoUDHC. (2015). *Urban Legal Cadastre Standard.* Technical report, Ministry of Urban Development, Housing and Construction, Addis Ababa, Ethiopia.

Müller, J., Gärtner-Roer, I., Thee, P., & Ginzler, C. (2014). Accuracy assessment of airborne photogrammetrically derived high-resolution digital elevation models in a high mountain environment. *ISPRS Journal of Photogrammetry and Remote Sensing, 98*, 58-69.

Mulugeta, E. (2018). Planimetric accuracy of digital spatial data for large scale mapping: the case study in Dire Dawa city, Ethiopia.

NGA. (2021, April 2). *Wikipedia*. Retrieved from https://en.wikipedia.org/wiki/Earth_Gravitational_Model.

NSSDA. (1998). Geospatial positioning accuracy standards Part 3: National standard for spatial data accuracy., (p. 1).

Pavlis, N. K., Holmes, S. A., Kenyon, S. C., & Factor, J. K. (2012). The development and evaluation of the Earth Gravitational Model 2008. *Journal of geophysical research, 117*, 1-38.

Reha Metin Alkan, Ibrahim Murat Ozulu, & Veli ilçi. (2016, May 2–6). Web-based GNSS data processing services as an alternative to conventional processing techniques. Turkey: Ibrahim Murat Ozulu.

Robert., A., Fowler, O.L.S., C.S.T., & C.E.T. (1997). *Concepts and applications of digital orthophotos.*

Sarhat Adam. (2017, July). The use of online and offline processing tools to improve the precision of a GPS passive station. *ResearchGate*.

Sharlene pillay. (2015, June). *Aerial photogrammetry.* Ethekwini municipality: Surveying and land information department.

Shingare, & Kale. (2013). Review on digital elevation model. *International journal of modern engineering research (IJMER)*.

Survey, U. G. (1998). *National Standard for Spatial Data Accuracy.* Federal Geographic Data Committee. Virginia: FGDC-STD.

Svetoslav. (2015). Accuracy evaluation and quality control of digital orthomap-sheets. *Wisdom of the ages to the challenges of the modern world*.

Takaku, J., Tadono, T., Tsutsui, K., & Ichikawa, M. (2016). Validation of 'AW3D' global DSM generated from ALOS. *ISPRS Annals of the Photogrammetry, Remote Sensing and Spatial Information Sciences, 3*, 25-31.

Tomaz Podobnikar. (2009). Methods for visual quality assessment of a digital terrain model. *Surveys and perspectives integrating environment and society*.

Toutin, T. (2002). Impact of terrain slope and aspect on radargrammetric DEM accuracy. *ISPRS Journal of Photogrammetry & Remote Sensing*, *57*, 228– 240. Retrieved from www.elsevier.com/locate/isprsjprs

Uuemaa, E., Ahi, S., Montibeller, B., Muru, M., & Kmoch, A. (2020, October 23). Vertical Accuracy of Freely Available Global Digital Elevation Models (ASTER, AW3D30, MERIT, TanDEM-X, SRTM, and NASADEM). *MDPI*. Retrieved from http://creativecommons.org/licenses/by/4.0/

Zinabu Getahun Sisay; Bedada Tulu; Gessesse Birhanne. (2017, June 17). Feature orientation and positional accuracy assessment of digital orthophoto and line map for large scale mapping: The case study on Bahirdar city. *The international archives of the photogrammetry, remote sensing and spatial information sciences, XLII-1/W1*, 6–9.

APPENDIX

Appendix A: Raw static GPS data

Name	Date modified	Type	Size
29062181.19n	8/21/2019 8:30 PM	19N File	14 KB
29062181	8/21/2019 8:30 PM	19O File	313 KB
29062181.t02	8/6/2019 9:12 AM	T02 File	62 KB
29062182.19n	8/21/2019 8:28 PM	19N File	16 KB
29062182	8/21/2019 8:28 PM	19O File	346 KB
29062182.t02	8/6/2019 11:38 AM	T02 File	71 KB
29062184.19n	8/21/2019 8:20 PM	19N File	16 KB
29062184	8/21/2019 8:20 PM	19O File	171 KB
29062184.t02	8/6/2019 1:10 PM	T02 File	39 KB
29062185.19n	8/21/2019 8:23 PM	19N File	11 KB
29062185	8/21/2019 8:23 PM	19O File	204 KB
29062185.t02	8/6/2019 2:39 PM	T02 File	42 KB
47102180.19n	8/21/2019 9:15 PM	19N File	16 KB
47102180	8/21/2019 9:15 PM	19O File	398 KB
47102180.t02	8/6/2019 10:06 AM	T02 File	78 KB
47102181.19n	8/21/2019 9:18 PM	19N File	16 KB
47102181	8/21/2019 9:18 PM	19O File	314 KB
47102181.t02	8/6/2019 12:31 PM	T02 File	63 KB
47102182.19n	8/21/2019 9:11 PM	19N File	11 KB
47102182	8/21/2019 9:11 PM	19O File	285 KB
47102182.t02	8/6/2019 2:27 PM	T02 File	56 KB
47932180.19n	8/21/2019 9:07 PM	19N File	42 KB
47932180	8/21/2019 9:07 PM	19O File	1,495 KB
47932180.t02	8/6/2019 2:51 PM	T02 File	285 KB
adis2180.19d_2	8/24/2019 11:41 AM	19D_2 File	872 KB

Appendix B: Raw RTK data collected through Trimble 5800 GPS

Checkpoints	Easting	Northing	Elevation	code
n1	811638.902	1063021.309	1159.793	tvt1
n2	811400.619	1063106.186	1152.14	tvt2
n3	811300.52	1063141.414	1146.891	tvt3
n4	811364.234	1063326.275	1143.413	tvt4
n5	811430.576	1063304.733	1143.878	tvt5
n6	811433.279	1063316.094	1144.01	fence1
n7	811501.959	1063292.937	1144.35	fence2
n8	811551.167	1063276.814	1146.073	school1
n9	811655.272	1063242.073	1149.069	school2
n10	811668.06	1063280.124	1145.314	school3
n11	811668.942	1063268.708	1148.2	church1
n12	811661.158	1063240.138	1148.784	church2
n13	811704.976	1063214.347	1150.519	tvt6
n14	811564.465	1063313.265	1143.911	schoo4
n15	811541.074	1063410.378	1142.553	fence3
n16	811471.995	1063434.303	1141.577	fence4
n17	811464.455	1063438.706	1141.546	kgschool1
n18	811456.501	1063414.566	1143.067	kgschool2
n19	811399.85	1063434.014	1141.495	kgschool3
n20	811407.477	1063458.17	1141.535	kgschool4
n21	811466.273	1063447.657	1141.553	RC01
n22	811411.777	1063466.568	1141.12	RC02
n23	811388.928	1063474.526	1140.812	RC03
n24	811326.803	1063495.949	1140.051	RC04
n25	811304.882	1063503.513	1139.814	RC05
n26	811159.359	1063553.998	1140.143	RC06
n27	811137.868	1063561.482	1140.293	RC07
n28	811017.311	1063603.304	1141.031	RC08
n29	811363.189	1063497.381	1139.883	fence5
n30	811398.549	1063484.731	1141.382	fence6

n31	811435.705	1063595.582	1137.28	fence7
n32	811422.775	1063711.438	1134.645	RC10
n33	811433.016	1063663.972	1135.916	RC11
n34	811441.941	1063622.918	1136.978	RC12
n35	811445.54	1063606.279	1137.395	RC13
n36	811448.085	1063594.261	1137.694	RC14
n37	811454.113	1063566.452	1138.432	RC15
n38	811460.448	1063537.287	1139.185	RC16
n39	811467.494	1063504.707	1140.005	RC17
n40	811474.193	1063473.673	1140.758	RC19
n41	811488.118	1063440.162	1141.729	RC20
n42	811509.439	1063432.775	1141.865	RC21
n43	811540.503	1063421.993	1142.036	RC22
n44	811571.356	1063411.324	1142.237	RC23
n45	811583.099	1063407.555	1142.318	RC24
n46	811603.116	1063402.033	1142.389	RC25
n47	811635.902	1063393.685	1142.511	RC26
n48	811649.975	1063389.941	1142.63	RC27
n49	811665.161	1063385.179	1142.709	RC28
n50	811677.703	1063380.786	1142.735	RC29
n51	811740.806	1063358.424	1143.059	RC30
n52	811717.815	1063252.5	1149.4	church3

Appendix C: Leica Geo-office report of post-processed static GPS data

- when it has to be right Leica Geosystems

Results - Baseline

cp002 - EMA

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: cp002	Rover: EMA
Receiver type / S/N:	Unknown / 4851162053	Unknown / 4913168810
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.6924 m	1.6544 m
Initial coordinates:		
Latitude:	9° 36' 54.58034" N	9° 36' 12.39680" N
Longitude:	41° 50' 15.82661" E	41° 50' 25.19637" E
Ellip. Hgt:	1142.3581 m	1186.3862 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:cp002		Rover:EMA
Coordinates:			
Latitude:	9° 36' 54.58034" N		9° 36' 12.39677" N
Longitude:	41° 50' 15.82661" E		41° 50' 25.19635" E
Ellip. Hgt:	1142.3581 m		1186.3861 m
Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0001 m	Sd. Lon: 0.0001 m	Sd. Hgt: 0.0004 m
	Posn. Qlty: 0.0002 m	Sd. Slope: 0.0001 m	

- when it has to be right Leica Geosystems

Results - Baseline

triangle - EMA

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: triangle	Rover: EMA
Receiver type / S/N:	Unknown / 4851162053	Unknown / 4913168810
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.6924 m	1.6544 m
Initial coordinates:		
Latitude:	9° 36' 38.10704" N	9° 36' 12.39680" N
Longitude:	41° 50' 18.22280" E	41° 50' 25.19637" E
Ellip. Hgt:	1151.3459 m	1186.3862 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:triangle		Rover:EMA
Coordinates:			
Latitude:	9° 36' 38.10704" N		9° 36' 12.39678" N
Longitude:	41° 50' 18.22280" E		41° 50' 25.19636" E
Ellip. Hgt:	1151.3459 m		1186.3844 m
Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0002 m	Sd. Lon: 0.0002 m	Sd. Hgt: 0.0005 m
	Posn. Qlty: 0.0003 m	Sd. Slope: 0.0002 m	

- when it has to be right Leica Geosystems

Results - Baseline

Meskelegna - EMA

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: Meskelegna	Rover: EMA
Receiver type / S/N:	Unknown / 4913168858	Unknown / 4913168810
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.7825 m	1.6544 m
Initial coordinates:		
Latitude:	9° 36' 33.21181" N	9° 36' 12.39680" N
Longitude:	41° 50' 52.34019" E	41° 50' 25.19637" E
Ellip. Hgt:	1162.8389 m	1186.3862 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:Meskelegna	Rover:EMA
Coordinates:		
Latitude:	9° 36' 33.21181" N	9° 36' 12.39681" N
Longitude:	41° 50' 52.34019" E	41° 50' 25.19636" E
Ellip. Hgt:	1162.8389 m	1186.3843 m

Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0001 m	Sd. Lon: 0.0002 m	Sd. Hgt: 0.0005 m
	Posn. Qlty: 0.0002 m	Sd. Slope: 0.0001 m	

- when it has to be right Leica Geosystems

Results - Baseline

Meskelegna - MA station

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: Meskelegna	Rover: MA station
Receiver type / S/N:	Unknown / 4913168858	Unknown / 4851162053
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.7825 m	1.7024 m
Initial coordinates:		
Latitude:	9° 36' 33.21181" N	9° 36' 16.71054" N
Longitude:	41° 50' 52.34019" E	41° 51' 09.56583" E
Ellip. Hgt:	1162.8389 m	1160.3170 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:Meskelegna	Rover:MA station
Coordinates:		
Latitude:	9° 36' 33.21181" N	9° 36' 16.71053" N
Longitude:	41° 50' 52.34019" E	41° 51' 09.56583" E
Ellip. Hgt:	1162.8389 m	1160.3198 m

Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0001 m	Sd. Lon: 0.0002 m	Sd. Hgt: 0.0006 m
	Posn. Qlty: 0.0002 m	Sd. Slope: 0.0002 m	

- when it has to be right Leica Geosystems

Results - Baseline

cp002 - papa

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: cp002	Rover: papa
Receiver type / S/N:	Unknown / 4851162053	Unknown / 4913168858
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.6924 m	1.4761 m
Initial coordinates:		
Latitude:	9° 36' 54.58034" N	9° 36' 17.37149" N
Longitude:	41° 50' 15.82661" E	41° 50' 01.74160" E
Ellip. Hgt:	1142.3581 m	1174.8695 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:cp002		Rover:papa
Coordinates:			
Latitude:	9° 36' 54.58034" N		9° 36' 17.37150" N
Longitude:	41° 50' 15.82661" E		41° 50' 01.74161" E
Ellip. Hgt:	1142.3581 m		1174.8636 m
Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0003 m	Sd. Lon: 0.0003 m	Sd. Hgt: 0.0010 m
	Posn. Qlty: 0.0004 m	Sd. Slope: 0.0003 m	

- when it has to be right Leica Geosystems

Results - Baseline

triangle - papa

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: triangle	Rover: papa
Receiver type / S/N:	Unknown / 4851162053	Unknown / 4913168858
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.6924 m	1.4761 m
Initial coordinates:		
Latitude:	9° 36' 38.10704" N	9° 36' 17.37149" N
Longitude:	41° 50' 18.22280" E	41° 50' 01.74160" E
Ellip. Hgt:	1151.3459 m	1174.8695 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:triangle		Rover:papa
Coordinates:			
Latitude:	9° 36' 38.10704" N		9° 36' 17.37151" N
Longitude:	41° 50' 18.22280" E		41° 50' 01.74162" E
Ellip. Hgt:	1151.3459 m		1174.8728 m
Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0002 m	Sd. Lon: 0.0003 m	Sd. Hgt: 0.0008 m
	Posn. Qlty: 0.0004 m	Sd. Slope: 0.0002 m	

- when it has to be right Leica Geosystems

Results - Baseline

triangle - Bridge

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: triangle	Rover: Bridge
Receiver type / S/N:	Unknown / 4851162053	Unknown / 4913168858
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.6924 m	1.4621 m
Initial coordinates:		
Latitude:	9° 36' 38.10704" N	9° 35' 58.53016" N
Longitude:	41° 50' 18.22280" E	41° 50' 53.02445" E
Ellip. Hgt:	1151.3459 m	1168.7666 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:triangle	Rover:Bridge
Coordinates:		
Latitude:	9° 36' 38.10704" N	9° 35' 58.53040" N
Longitude:	41° 50' 18.22280" E	41° 50' 53.02442" E
Ellip. Hgt:	1151.3459 m	1168.7732 m

Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0007 m	Sd. Lon: 0.0006 m	Sd. Hgt: 0.0011 m
	Posn. Qlty: 0.0009 m	Sd. Slope: 0.0005 m	

- when it has to be right Leica Geosystems

Results - Baseline

cp002 - 7th

Project Information

Project name:	Nati Thesis
Date created:	08/26/2019 04:18:49
Time zone:	3h 00'
Coordinate system name:	WGS1984 (2)
Application software:	LEICA Geo Office 7.0
Processing kernel:	PSI-Pro 2.0
Processed:	08/26/2019 04:45:33

Point Information

	Reference: cp002	Rover: 7th
Receiver type / S/N:	Unknown / 4851162053	Unknown / 4913168858
Antenna type / S/N:	TRM5800 / -	TRM5800 / -
Antenna height:	1.6924 m	1.5622 m
Initial coordinates:		
Latitude:	9° 36' 54.58034" N	9° 36' 43.60211" N
Longitude:	41° 50' 15.82661" E	41° 50' 03.50140" E
Ellip. Hgt:	1142.3581 m	1150.9670 m

Processing Parameters

Parameters	Selected	Used	Comment
Cut-off angle:	15°	15°	
Ephemeris type (GPS):	Broadcast	Broadcast	
Ephemeris type (GLONASS):	Broadcast	Broadcast	
Solution type:	Automatic	Phase: all fix	
GNSS type:	Automatic	GPS	
Frequency:	Automatic	Automatic	
Fix ambiguities up to:	80 km	80 km	
Min. duration for float solution (static):	5' 00"	5' 00"	
Sampling rate:	Use all	15	
Tropospheric model:	Hopfield	Hopfield	
Ionospheric model:	Automatic	Computed	
Use stochastic modelling:	Yes	Yes	
Min. distance:	8 km	8 km	
Ionospheric activity:	Automatic	Automatic	

Satellite Selection

Manually disabled GPS satellites (PRNs):	None
Manually disabled GLONASS satellites (Slot Id):	None

Final Coordinates

	Reference:cp002	Rover:7th
Coordinates:		
Latitude:	9° 36' 54.58034" N	9° 36' 43.60214" N
Longitude:	41° 50' 15.82661" E	41° 50' 03.50141" E
Ellip. Hgt:	1142.3581 m	1150.9681 m

Solution type:	Phase: all fix		
GNSS type:	GPS		
Frequency:	L1 and L2		
Ambiguity:	Yes		
Quality:	Sd. Lat: 0.0002 m	Sd. Lon: 0.0002 m	Sd. Hgt: 0.0006 m
	Posn. Qlty: 0.0003 m	Sd. Slope: 0.0002 m	

Appendix D: Online (AUSPOS) report of post-processed static GPS data (Double click the link file below)

AUSPOS GPS Processing Report

August 25, 2019

This document is a report of the GPS data processing undertaken by the AUSPOS Online GPS Processing Service (version: AUSPOS 2.3) . The AUSPOS Online GPS Processing Service uses International GNSS Service (IGS) products (final, rapid, ultra-rapid depending on availability) to compute precise coordinates in International Terrestrial Reference Frame (ITRF) anywhere on Earth and Geocentric Datum of Australia (GDA) within Australia. The Service is designed to process only dual frequency GPS phase data.

An overview of the GPS processing strategy is included in this report.

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Appendix E: Accuracy of Static GPS measurement of selected Checkpoints.

Station Code	Coordinates derived from GPS		Coordinate derived from orthophoto		Residuals(Errors)	
	Easting	**Northing**	**Easting**	**Northing**	**Δ Easting**	**ΔNorthing**
Sebategn	811016.8342	1063605.185	811016.59	1063605.422	0.24424	-0.23699
Maingate	811390.1334	1063945.845	811390.027	1063945.647	0.10637	0.19801
Papa	810969.7985	1062798.223	810969.719	1062798.311	0.07951	-0.08771
Bridge	812539.4975	1062231.862	812539.37	1062231.657	0.12748	0.20542
MA	813039.6235	1062795.05	813039.487	1062794.761	0.13652	0.28852
Mesklgna	812509.768	1063298.056	812509.822	1063298.11	-0.05404	-0.05355
Triangle	811467.4437	1063439.941	811467.414	1063439.866	0.02971	0.07547
				MIN	0.02971	0.05355
				MAX	0.24424	0.28852
				Standard deviation	0.0201	0.0302
				RMSE	0.12872	0.18403
			RMSEr	**0.2246**	SQRT($RMSEx^2$ $+RMSEy^2$)	
			NSSDA at 95% confidence level	**0.3887**	RMSEr*1.7308	

Appendix F: Accuracy of Areal Features of twenty Sample Parcel corners.

Station Code	Coordinates derived from GPS		Coordinate derived from orthophoto		Residuals(Errors)	
	Easting	**Northing**	**Easting**	**Northing**	**Δ Easting**	**ΔNorthing**
pc1	811638.902	1063021.31	811638.93	1063021.15	-0.023	0.161
pc2	811303.52	1063139.41	811304.26	1063139.07	-0.738	0.343

pc3	811367.234	1063324.28	811367.42	1063323.96	-0.186	0.318
pc4	811433.576	1063302.13	811433.75	1063301.88	-0.172	0.25
pc5	811436.279	1063314.09	811437.07	1063313.92	-0.787	0.17
pc6	811504.959	1063291.94	811504.98	1063291.84	-0.025	0.102
pc7	811554.167	1063274.81	811554.99	1063274.58	-0.827	0.232
pc8	811658.072	1063240.07	811658.27	1063239.94	-0.196	0.137
pc9	811671.06	1063278.12	811671.17	1063277.81	-0.106	0.311
pc10	811671.942	1063266.71	811671.97	1063266.5	-0.029	0.208
pc11	811664.158	1063238.14	811664.55	1063237.82	-0.394	0.317
pc12	811707.076	1063212.15	811707.55	1063211.85	-0.476	0.301
pc13	811567.465	1063311.07	811567.63	1063310.83	-0.169	0.237
pc14	811543.074	1063408.18	811543.63	1063408	-0.56	0.182
pc15	811474.995	1063432.3	811475.56	1063432	-0.565	0.306
pc16	811466.455	1063436.71	811467.44	1063436.46	-0.982	0.245
pc17	811459.501	1063413.17	811460.03	1063412.97	-0.527	0.194
pc18	811403.005	1063432.01	811403.04	1063431.9	-0.032	0.119
pc19	811410.477	1063456.17	811410.97	1063455.91	-0.497	0.259
pc20	811720.815	1063250.3	811721.15	1063249.92	-0.338	0.379
MIN					-0.982	0.102
MAX					-0.023	0.379
Standard deviation					0.276	0.0377
RMSE					0.32	0.21
RMSEr				**0.38**	SQRT(RMSEx^2 +RMSEy^2)	
NSSDA at 95% confidence level				**0.66**	RMSEr*1.7308	

Appendix G: Ground control point data (reference data)

GCP	Easting(Adindan)	Northing(Adindan)	Orthometric height
1	804991	1065941	1102.887
2	807840.4	1066003	1101.505
3	808950.2	1065912	1112.143

4	814937	1065040	1143.307
5	812827.3	1065903	1115.976
6	814071.1	1065033	1135.514
7	809016.7	1064993	1130.388
8	807837.4	1064993	1121.145
9	809965.3	1064974	1135.424
10	806982.7	1065046	1111.18
11	805010.9	1064983	1119.378
12	804131.1	1065040	1117.54
13	809012.6	1064057	1149.429
14	810029.7	1064117	1151.792
15	811753.6	1064046	1147.659
16	813049.4	1064040	1144.601
17	813915.9	1063891	1157.917
18	815040.5	1063908	1155.383
19	810961.9	1065057	1134.977
20	812001.9	1065028	1125.539
21	813225.9	1065085	1130.671
22	808968.9	1063080	1170.335
23	806850.8	1063869	1126.243
24	805955.5	1064039	1131.244
25	804907	1063041	1153.107
26	804022.6	1062928	1150.954
27	803994.6	1064038	1133.865
28	804994.2	1064162	1133.987
29	806008	1065013	1114.746
30	806008.6	1065968	1101.459
31	810989.7	1067025	1099.526
32	813829	1063157	1173.101
33	814100.1	1061754	1203.721
34	814907.7	1062011	1193.774
35	813017.3	1063107	1171.491

36	813006.5	1058973	1297.146
37	813925.8	1060099	1252.457
38	814991.7	1059005	1325.503
39	814000.9	1058991	1305.529
40	816141.1	1058922	1332.474
41	814910.7	1058463	1354.335
42	815056.2	1060916	1326.647
43	812833.3	1061964	1188.622
44	812054.7	1062032	1192.02
45	810955.4	1062004	1193.994
46	817014.8	1062106	1208.912
47	816115.7	1062039	1192.137
48	816007.3	1063012	1186.046
49	815132.1	1062905	1178.763
50	814038.7	1061027	1216.497
51	813003.6	1060973	1201.981
52	815128.1	1059958	1260.857
53	807884.8	1064012	1140.013
54	810840.8	1064113	1149.336
55	805318.5	1065561	1109.692
56	808445.4	1065985	1105.896
57	813457.6	1064313	1147.383
58	804945.4	1064577	1127.197
59	811946.5	1064065	1147.912
60	814093.3	1063562	1166.987
61	804834.9	1062853	1155.004
62	804335.8	1062925	1152.643
63	804763.1	1064213	1133.166
64	805792.1	1064989	1117.102
65	806079.4	1065619	1103.756
66	814231.5	1061829	1203.964
67	811414.4	1063474	1160.644

68	813046.5	1062104	1190.56
69	815140.5	1062496	1186.63
70	813096.5	1061053	1203.581

Appendix H: Metadata of Dire Dawa city aerial imagery

Name	Dire Dawa City Aerial Imagery 2012
Abstract	Dire Dawa City Aerial photography Flown on March 2012
Date of capture	March 2012
Data type	Raster
Native Data format	TIFF
Update frequency	No update
Positional Accuracy	The positional accuracy of this image after Aerial Triangulation is X=0.129, Y=0.133 and Z=0.202
Accuracy statement	The accuracy of Dire Dawa City Aerial Imagery has been approved by the Ethiopian Mapping Agency for its intended purpose.
Resolution/GSD	15 cm
Data History	Flown by Information Networking Security Agency Camera: ICAROS IDM 200 Medium Format Camera Pixel size: Along track; Across track:8984 Physical pixel sisze:6 micron Focal length:55.3 Overlap: End lap 70%;Side lap 40% Aircraft: DHC 6,Lever arms'=-0.198m,y=-0.57235m,y=-2.02m Boresight Angle:
Additional information	Imagery, AT, and DTM of this city have been delivered to Ethiopia Mapping Agency (EMA). DLM (Digital Line Map) has been prepared using this data as input.
Coordinate system	UTM, Datum WGS 84 and Adindian Clarke 1880,Zone 37
Point of contact	Information Networking Security Agency, Geospatial Information Security Directorate **E-mail**:tazergebre@insa.gov.et Phone:+251113204005
Meta Data Author	Mifta Zeynu E-mail: Miztyo@gmail.com Phone:+251936672673

Appendix I: Vertical accuracy assessment result

Checkpoints	Ground Elevation	DEM Elevation(Z)	Δ Elevations(Z)	$(\Delta z)^2$
GC1	1102.887	1106.397	-3.51	12.320
GC2	1101.505	1103.653	-2.15	4.613
GC3	1112.143	1114.650	-2.51	6.285
GC4	1143.307	1139.395	3.91	15.301
GC5	1115.976	1115.548	0.43	0.184
GC6	1135.514	1135.085	0.43	0.184
GC7	1130.388	1132.393	-2.01	4.021
GC8	1121.145	1123.947	-2.80	7.849
GC9	1135.424	1138.728	-3.30	10.917
GC10	1111.18	1109.583	1.60	2.550
GC11	1119.378	1121.240	-1.86	3.467
GC12	1117.54	1120.560	-3.02	9.119
GC13	1149.429	1150.868	-1.44	2.072
GC14	1151.792	1154.643	-2.85	8.131
GC15	1147.659	1149.772	-2.11	4.464
GC16	1144.601	1145.947	-1.35	1.812
GC17	1157.917	1158.547	-0.63	0.397
GC18	1155.383	1160.287	-4.90	24.050
GC19	1134.977	1137.242	-2.27	5.131
GC20	1125.539	1129.443	-3.90	15.237
GC21	1130.671	1132.406	-1.74	3.011
GC22	1170.335	1172.587	-2.25	5.072
GC23	1126.243	1132.104	-5.86	34.354
GC24	1131.244	1135.687	-4.44	19.738
GC25	1153.107	1154.000	-0.89	0.797
GC26	1150.954	1153.597	-2.64	6.984
GC27	1133.865	1135.800	-1.94	3.744
GC28	1133.987	1135.789	-1.80	3.246

GC29	1114.746	1118.107	-3.36	11.293
GC30	1101.459	1104.732	-3.27	10.713
GC31	1099.526	1103.006	-3.48	12.112
GC32	1173.101	1175.679	-2.58	6.645
GC33	1203.721	1204.277	-0.56	0.310
GC34	1193.774	1195.856	-2.08	4.334
GC35	1171.491	1174.456	-2.96	8.791
GC36	1297.146	1282.500	14.65	214.505
GC37	1252.457	1274.387	-21.93	480.945
GC38	1325.503	1316.664	8.84	78.120
GC39	1305.529	1307.684	-2.15	4.642
GC40	1332.474	1308.857	23.62	557.743
GC41	1354.335	1333.081	21.25	451.751
GC42	1326.647	1310.509	16.14	260.450
GC43	1188.622	1185.061	3.56	12.679
GC44	1192.02	1193.403	-1.38	1.912
GC45	1193.994	1195.862	-1.87	3.488
GC46	1208.912	1214.125	-5.21	27.172
GC47	1192.137	1197.732	-5.59	31.301
GC48	1186.046	1185.036	1.01	1.020
GC49	1178.763	1180.435	-1.67	2.795
GC50	1216.497	1218.282	-1.78	3.185
GC51	1201.981	1205.133	-3.15	9.936
GC52	1260.857	1269.717	-8.86	78.496
GC53	1140.013	1140.150	-0.14	0.019
GC54	1149.336	1153.013	-3.68	13.523
GC55	1109.692	1110.822	-1.13	1.277
GC56	1105.896	1103.782	2.11	4.471
GC57	1147.383	1187.279	-39.90	1591.656
GC58	1127.197	1130.349	-3.15	9.937
GC59	1147.912	1151.106	-3.19	10.203
GC60	1166.987	1168.460	-1.47	2.169

GC61	1155.004	1157.856	-2.85	8.135
GC62	1152.643	1154.150	-1.51	2.271
GC63	1133.166	1136.084	-2.92	8.517
GC64	1117.102	1120.147	-3.04	9.271
GC65	1103.756	1107.488	-3.73	13.930
GC66	1203.964	1204.866	-0.90	0.814
GC67	1160.644	1162.200	-1.56	2.421
GC68	1190.56	1190.553	0.01	0.000
GC69	1186.63	1186.688	-0.06	0.003
GC70	1203.581	1204.620	-1.04	1.080
			Sum	**1086.987**
			Average	**16.469**
RMSEz				**4.058**
NSSDA at 95% confidence level				**7.954**