

Optimal Scenario Development and Sensitivity Analysis Methodology for Multi-Standard Geospatial Positional Accuracy Testing in Varied Topography: Evidence from Ethiopian Case Studies

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Abstract

Ethiopia uses photogrammetry for land registration and planning, but in-situ calibration is required due to factors affecting geometric accuracy. This study aimed to develop optimal scenarios and a sensitivity analysis framework for multi-standard geospatial positional accuracy testing of photogrammetric-derived orthophotos across varied topographies in Ethiopia. GNSS static measurements, considered true positions, were used to evaluate orthophoto accuracy, and the data were least-squares adjusted using Leica Geo-Office. Three scenarios were designed based on the number of checkpoints (CPs), 10, 15, and 20, considering both the number and spatial distribution of CPs across three cities with diverse topography. Scenario development and sensitivity analysis were guided by multi-standard principles. Results showed that positional accuracy was not highly sensitive to variations in CP numbers or distribution. Specifically, the combined RMSE in easting and northing for 10, 15, and 20 CPs were $\pm 36, \pm 40$, and $\pm 32; \pm 38, \pm 40$, and ± 33 ; and $\pm 39, \pm 40$, and ± 35 cm in Bahir Dar, Harer, and Debre Markos, respectively. Coordinate differences between orthophotos and GNSS measurements appeared as systematic shifts rather than random errors. While increasing CP from 10 to 20 slightly reduced RMSE deviations among scenarios and case studies, achieving optimal accuracy depended more on selecting representative CP locations, such as sharp or visible corners of manmade features, and accounting for topographic variations. Overall, sensitivity analysis combined with multi-standard approaches provides a robust and practical framework for assessing positional accuracy, ensuring that photogrammetric-derived geospatial data are reliable and suitable for planning, development, and mapping applications across diverse terrains.

1. Introduction

Ethiopia is one of the most developing countries in the world with rapid urbanization. When we look at back to the history, most of major Ethiopian cities including the capital city, Addis Ababa, emerged as an urban center without significant planning and urban cadastral intervention. As a response to the current urban land development and management problems induced by rapid urbanization, the Ethiopian government has established legal frameworks and institutional set-up to modernize land administration through legal cadaster. For instance, Ethiopian Urban Legal Cadastral Standard No.-03/2015 (MoUI, 2015) stated that the spatial component (parcel map) of the data should be obtained using ground surveying, aerial photography and high resolution satellite imagery. When extracting cadastral features from photogrammetric techniques, positional verification is important through following scientific steps before using the intended applications.

Nowadays, continuous advancements have been conducting on digital photogrammetric science especially on sensor (types and quality) platform (types and stability). The reason for this advancement is digital photogrammetry started to be used for all mapping applications, such as cadastral, urban planning, utility and irrigation site maps. According to literatures, the main motivations for using this technique are speed and cost of mapping for both rural and urban areas and seamless coverage especially for cadastral renovation and utility mapping (David Siriba, 2009). In order to realize the production of large scale mapping, calibration by in-situ measurement is critical due to the fact that the photogrammetric surveying products are affected by different factors such as topographic variations/relief displacement, camera and sensor orientation and ground control points (GCP) distribution across experimental site, earth's curvature, software used and other error sources introduced by computational procedures such as rectification due to height variation. Other contributing error factors include the characteristics and calibration of equipment used for image capture such as the camera and/or scanner (USGS, 2023). Photogrammetric processing variables, estimation of orientation parameters, and correction of lens distortion are crucial factors that also affect the accuracy of any spatial products in the field of photogrammetry (DENG et.al., 2025; Gond et al., 2025). Furthermore, stationary terrain features will change their position caused by a change in viewing a position and orientation specifically in photogrammetry and Light Detection and Ranging (LIDAR) technologies. Similarly, Nwilo et al.(2022) found that Google Earth imagery exhibits geo-registration problems and large horizontal positional errors.. Due to such factors, orthophotos and digital elevation models (DEM) are not generated with equal accuracy level (J. Greenfeld, 200; Rabiw & Waziri, 2014). In general, the accuracy and quality of the aerial surveying product varies based on the accuracy of the source data and checkpoint sampling design and strategies (Congalton & Green, 2008; Sisay et al., 2017). According to (Congalton & Green, 2008), testing positional accuracy is essential because reliable decisions depend on maps that are accurate or have known accuracy. Accuracy assessment helps evaluate map quality, identify and correct errors, and compare different techniques, algorithms, or analysts to determine the most effective approach.

Currently, Ethiopia has adopted an integrated surveying approach that combines; ground-based, aerial and high resolution satellite products to support land registration urban and rural planning, irrigation site development, and any other spatial oriented projects across various sectors (MoUI, 2015). The(DEV, 2025) strategy emphasizes the role of geospatial technologies in climate risk analysis, tourism hotspot identification, and logistical support, whereas Rajabifard(2019) stresses their contribution to enhancing the achievement of the Sustainable Development Goals (SDGs). In this case, photogrammetric and satellite image-derived products are very important and require in-situ ground calibration and validation against both international and national standards to ensure their accuracy for the intended purposes. However, spatial data accuracy varies significantly across sectors in Ethiopia, primarily due to the lack of appropriately designed accuracy assessment methodologies and a weak Spatial Data Infrastructure (SDI) framework. Apart from this, topographic variation within the experimental sites is one of the triggering factors for error propagation in geospatial products. Ethiopia's topography, which ranges from approximately 120 meters to 4,620 meters above sea level, may significantly affect positional accuracy. For example, the geometric structure and positional alignment of spatial datasets often differ among land registration, urban planning, and city administration systems. Addressing these inconsistencies through the development of comprehensive accuracy testing methodologies is essential for improving land management operations, reducing duplication of effort, and promoting coherent national planning strategies.

This study aims to address gaps in geospatial data accuracy by optimizing multi-standard designs and conducting sensitivity analyses using precise GNSS reference datasets, guided by international and national analytical frameworks.

2. Analytical Framework

This study has applied multiple international positional accuracy standards, including ASPRS (2023), NMAS (1947), FGDC/NSSDA (1998), STANAG 2215 (2010), and FEMA (2003), alongside relevant Ethiopian national standards, to analyze, evaluate and compare scenario-based positional accuracy results. Country’s guidelines and standards differ in their sample-size and methodological recommendations, depending on the task’s goals and on whether data errors are systematic or random. In this context, while testing horizontal and vertical positional accuracy, the major geospatial accuracy standards differ widely in how they address sample-count requirements. Early standards such as(NMAS, 1947) rely on allowable percentages of failing points relative to map scale instead of fixing the number of samples for positional testing of any geo-spatial products. On one hand,(Greenwalt, C. R. and Shultz, M. E. 1992) explain the statistical theory for sampling without prescribing a universal number of samples. Furthermore,(ASPRS, 2023) large-scale map standard similarly focuses on methodology and reporting instead of fixing the number of CPs. In contrast,(NSSDA/FGDC, 1998; NSSDA/FGDC, 2002) introduces the commonly used requirement of a minimum of 20 well-distributed independent CPs for statistical optimization. Furthermore,(FEMA, 2003) generally adopts NSSDA’s 20-point principles.

For tasks demanding height precision,(ASPRS, 2025) LiDAR guidelines and(FEMA, 2003; NDEP, 2004) framework do not impose a single sample count but instead define explicit accuracy-test categories (Full, Spot, and Consolidated Validation Accuracy), the latter used only when ≥ 40 CPs can be consolidated. Overall, contemporary practice recognizes that while minimum counts (such as NSSDA’s 20 points) provide a baseline, appropriate sampling density depends on terrain, land cover, and project-specific accuracy requirements.(Newby, P. R. (1992)) also states that instead of fixing the number of CPs, it stresses sound sampling design, independence of CPs, and RMSE-based reporting suited to project needs. According to(U.SCECW, 2012) Data Quality Objectives (DQOs) should be developed based on specific project requirements, rather than relying on standardized, one-size-fits-all number of samples.

In other words,Ariza López & Atkinson Gordo, Alan David, 2008; NJUG,1988; Newby, P. R. (1992) similarly recommended a sample size of approximately 50 checkpoints for positional and accuracy testing in geospatial products, rather than adhering to a fixed minimum such as the 20 checkpoints required by (NSSDA/FGDC, 1998; NSSDA/FGDC, 2002). Furthermore, unlike NSSDA’s baseline of 20 checkpoints,STANAG 2215, (2010) implied sample requirement (approximately 167 points) is substantially larger, reflecting its military cartographic emphasis on comprehensive positional testing across map elements.

On the other hand, specifically for positional control, ASPRS (2023) asserts that the sampling size and their distributions are dependent on topography and area of experimental site. Accordingly, the recommended number of checkpoints for positional accuracy verification increases with the spatial extent of the experimental site, ranging from a minimum of 20 checkpoints for areas up to 500 km² to 60 checkpoints for areas between 2,251 and 2,500 km² when clearly identifiable points are used (Table 1).

Table 1
Recommended Number of CPs by ASPRS, 2023

Area (km ²)	Recommended No. of CPs	Area (km ²)	Recommended No. of CPs
≤ 500	20	1501–1750	45
501–750	25	1751–2000	50
751–1000	30	2001–2250	55
1001–1250	35	2251–2500	60
1251–1500	40	1501–1750	45

In contrary to the(ASPRS, 2023) standard above, the geographical extent of the experimental site isn’t the only factor for determination of sample size, but also the spatial distribution of the sample can condition the validity of a statistical sampling assessment. A bad spatial distribution affects the representativeness of the sample. This means that the sample does not capture adequately the structure of the population being sampled, resulting in an erroneous estimation(Ariza López & Atkinson Gordo, Alan David, 2008); (ASPRS, 2023)). In support of this,PAAMS, 2026) provide explicit criteria for a suitable spatial distribution sampling checkpoints. In some cases the need for an agreement between the producer and the user (UNICEF, 2002). When the experiment is conducted in a rectangular area, an ideal distribution of test points allows for at least 20% of the points to be located in each quadrant. Test points should be spaced at intervals of at least 10% of the diagonal distance across the rectangular area. The remaining 20% of the sample will be intensified as per the users’ interest (NSSDA/FGDC, 1998; NSSDA/FGDC, 2002).

Other perspective argues that due to the diverse user requirements for digital geospatial data and maps, it is not realistic to include statements that specify the spatial distribution of sampling checkpoints. Data and/or map producers must determine checkpoint location (NSSDA/FGDC, 1998; NSSDA/FGDC, 2002). This standard also explains that checkpoints may be distributed more densely in the vicinity of important features and more sparsely in areas that are of little or no interest. So far, the standard determines a minimum of 20 sample points are required to test positional accuracy with reliable statistical rigor. In considering the above disparities, ASPRS (2025) standard recommends that quantitative characterization and specification of the spatial distribution of checkpoints across the same project area, let alone across each land cover type within a project, will require significant additional research.

3. Literature Review

Several studies have been undertaken using the above international positional accuracy standards. However, certain gaps in knowledge, theory, and methodology have been identified. For instance, the works of Drobnjak and Bozic (2018) applied a single standard without providing a theoretical explanation for why Class A accuracy is required, and they used a high CP density without testing alternative CP configurations or reduced datasets. Moreover, the study by Mesas-Carrascosa et al. (2014) did not clarify how accuracy varies across terrain complexity and lacked a sensitivity analysis on increasing or decreasing CPs using an independent validation dataset, even though multiple standards were applied. Similarly, Mantey and Aduah (2022) used a relatively small

number of GNSS points as CPs, which may affect statistical robustness, but this issue was not discussed. Radojčić (2022) relied solely on a single standard and treated it as a fixed criterion without theoretical justification. Other recent studies (Liu et al., 2022; Sanz-Ablanedo et al., 2018; Zhang et al., 2022) also have methodological limitations, including over-reliance on RMSE as the primary accuracy metric, relatively controlled conditions on flat topography, insufficient use of independent validation datasets, and a lack of sensitivity and uncertainty analysis.

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Thus, based on the methodological and theoretical gaps identified in previous studies, several directions for future research emerge; one is on optimizing both the number and spatial distribution of CPs to balance accuracy and efficiency across different mapping platforms. Additionally, comparative studies and multi-standard accuracy assessments are needed to improve consistency and interpretability. Advancing methodology by implementing multi-standard and multi-platform accuracy evaluations, conducting sensitivity analyses on the number of control points, incorporating independent validation datasets, and applying advanced accuracy metrics beyond RMSE is critical to ensure robust and generalizable results including for modern Unmanned Aerial Vehicle (UAV) and GNSS-based mapping applications (Atik et al., 2025; Drobnjak & Bozic, 2018; Liu et al., 2022; Mantey & Aduah, 2022; Mesas-Carrascosa et al., 2014; Radojčić, 2022; Sanz-Ablanedo et al., 2018; Zhang et al., 2022).

In conclusion, key success factors for positional testing of any geospatial product include optimizing the number and distribution of CPs, using independent validation datasets, conducting multi-standard and multi-platform assessments, performing sensitivity analyses, applying advanced accuracy metrics beyond RMSE, accounting for terrain and platform differences, and ensuring efficiency and statistical robustness. Together, these practices ensure that geospatial products are accurately evaluated, reliable across diverse conditions, and statistically sound, providing confidence in their positional integrity for a wide range of applications.

Therefore, this study aims to address key gaps in the modeling and testing of optimal scenarios for geospatial data positional accuracy analysis by optimizing multi-standard design and sensitivity analyses against precise GNSS reference datasets. Specifically, it evaluates how variations in CP quantity, spatial distribution, and methodological use influence accuracy results across different terrains and mapping platforms. The findings are intended to provide geospatial experts and industry practitioners with insights into designing efficient and statistically robust CP network design across varied topography, particularly for GNSS-enabled photogrammetric applications.

4. Materials and Methods

4.1 Experimental Site

This study was conducted on three experimental sites purposely, having different topography characteristics and different geographical extent, although all the experimental sites are under 500 square kilometers. The first site is Debre Markos city, which is characterized by undulated terrain, situated at an altitude range of 1,933 to 2,852 meters above mean sea level (MSL), and having 192.3 km² area coverage. The second city is Bahir Dar, characterized by flat plain, situated at an altitude range of 1650 to 1886 meters above MSL, and having 362 km² area coverage. The third city is Harar, characterized by slightly undulating topography, and situated at an altitude range of 1836–1926 meters above MSL, and having 334 km² area coverage (Fig. 1).

4.2 Data Used

The data source for this study is a rectified aerial photograph, secondary Ground Control Points (GCP), and a Digital Elevation Model (DEM) acquired over the whole area of the above listed experimental sites. The rectified aerial photograph and DEM were generated by using the ArcInfo photogrammetric software platform. The photogrammetric surveying was conducted by the Information Network Security Agency (INSA) in 2011/12 with middle frame camera at 1:2,000 scale and 15 cm Ground Sample Distance (GSD). The reference secondary control point coordinates were observed for 12 hours in connection with the primary point (determined in 48 hours of GNSS observation) and their data was computed in connection with International GNSS Service (IGS) using LGO and Ashtech solutions software packages by the Ethiopian Mapping Agency (EMA, 2013). All the data sources in this study were geometrically registered to the UTM reference system (Zone – 37 N) for all experimental sites using the Adindan (Ethiopia) Plane horizontal datum.

4.3 Adopted International and National Accuracy Standards Framework

This study benchmarked (ASPRS, 2023; FEMA, 2003; NMAS, 1947; NSSDA/FGDC, 1998; NSSDA/FGDC, 2002; STANAG 2215, (2010)) as the primary positional accuracy standards. Together, these standards provide statistically rigorous, operationally relevant, and terrain-adaptive frameworks for evaluating CP network design and sensitivity analysis in GNSS-enabled photogrammetric workflows. Furthermore, ASPRS provides guidance for CP quantities, terrain classification, and Circular Error with 95 % confidence level (CE95), where $CE95 \approx 1.7308 \times RMSE_r$ for horizontal positional accuracy, an Linear Error (LE), where $LE95 \approx 1.9600 \times RMSE_z$ for vertical positional accuracy at the 95% confidence level (CL). These characteristics make ASPRS suitable for multi-scenario CP sensitivity modeling, consistent with this study's objective. The NSSDA/FGDC, 1998; NSSDA/FGDC, 2002) standards provide excellent guidance as a baseline model for CP sufficiency testing, sample design, estimation of sample size, and distribution, and applies the principles of CE and LE at the 95% CL. (STANAG 2215, (2010)) is well suited for strict statistical control in sensitivity analysis, also applying CE and LE at the 95% CL, while, FEMA (2003), is critical for vertical accuracy analysis in photogrammetric applications. Thus, the statistical principles and underlying assumptions of these four standards are consistent with this study's objectives and CP sampling behavior.

In addition to international standards, this study examines Ethiopian national accuracy assessment standards, specifically the Urban Legal Standard No. 03/2015 MoUI (2015), regarding the error budget limit of digital orthophotos in terms of RMSE. According to this standard, the error corresponds to two pixels at a Ground Sample Distance (GSD) of 15cm, regardless of the method used; however, the positional accuracy of the final output should not exceed

40cm at a scale of 1:2000. The vertical positional accuracy is $\pm 45\text{cm}$, which corresponds to three pixels. Another national standard, set by the Ethiopian Mapping Agency (EMA), specifies an accuracy of $\pm 30\text{cm}$ at a scale of 1:2000, which is the recommended scale for urban areas.

4.4 Scenario Development and Sampling Requirements

In order to determine the optimum sample size for orthophoto positional verification, the study has developed three scenarios based on the number of sampling checkpoints, their spatial distribution and topographic undulation characteristics for each experimental site. In this context, the first, the second and the third scenarios consist of 10, 15 and 20 sampling checkpoints (CPs) respectively, for each experimental site based on the adopted standards as shown in the Fig. 2. According to (ASPRS (2023)) the recommended number of checkpoints varies with the corresponding experimental site. The standard explicitly states that when the experimental site area varied by 250 in square kilometers, checkpoints are also varied by 5 in numbers simultaneously. Accordingly, this study developed scenarios with 10, 15 and 20 checkpoints to test how positional variation changes when the number of sampling checkpoints varied by increments of 5, starting from 10 while considering varied topography and multi-standard and sensitivity analysis.

Positional accuracy assessment is scientifically valid only when reference data are independent of the dataset being evaluated and are systematically selected to capture the spatial and feature variability of the study area. Accordingly, the checkpoints (CPs) in all scenarios and experimental cities exhibited the following characteristics:

- CPs were not used in dataset generation, georeferencing, calibration, or model training.
- CPs were representative of the study area and evenly distributed across the experimental sites.
- CPs accounted for terrain variation and key features, including road junctions, culverts, Dutch corners, and other locations with sharp geometric characteristics as shown in Fig. 3.

4.5 Measurement, Processing and Analysis

A minimum of One hour GNSS observation has been collected in static mode for each checkpoint and processed using LGO by using verified secondary GCP for the reference station with less than 3 km base-line distance, within each city. During the measurements, from 8 to 12 satellites, both GPS and GLONASS were visible, and Geometric Dilution of Precision (GDOP) varied between 1.2 and 2.8, which is considered acceptable for most mapping and engineering applications.

In the processing using the LGO software package, the translation parameters in x, y and z direction were 162, 12 and - 206 m, the rotation parameters in x, y and z were also 0, 0 and 0 m, the scale factor was 0.9996 and the projection was Universal Transverse Mercator (UTM) 37 N while converting WGS84 geodetic co- ordinates to Adindan geodetic co-ordinates. Following this, the three scenarios were tested in each three experimental site and the results were analyzed in terms of RMSE in x and y-direction independently and together based on the adopted standards as indicated in Fig. 4.

5. Results

This section presents the empirical findings derived from the analysis of the collected data from GNSS measurements, and extracted numerical results from orthophoto across the defined experimental scenarios. The results are organized to reflect the key components of the study, with emphasis on measurement precision, data accuracy, and comparative performance among scenarios.

5.1 GNSS Processing

With regard to data collection, two GNSS receivers were placed at the reference stations, which are known points, while the remaining two receivers were placed at each CPs location. In other words, two GNSS receivers were used for reference and two for CPs data collection. A total of 10 observation sessions and 20 baselines were computed in GNSS processing for each of the three scenarios. The baseline length between reference stations and the CPs location, as well as their spatial distribution across experimental site were considered in baseline selection for each scenario. The quality status of the reference stations indicates average errors of 6 mm, 7 mm and 8 mm, respectively at 95% confidence level. The precision of the GNSS processing results obtained with the LGO software is summarized in Table 2a, 2b and 2c below. The precision for GNSS measurement is expressed as position and height quality (P + HQ).

Table 2

a. The static GNSS coordinate results in meter and position and height quality (P + HQ) in millimeter (mm) obtained from LGO software for Bahir Dar city.

ID	E (m)	N(m)	H (m)	P+ HQ	ID	E (m)	N(m)	H (m)	P+ HQ
1	324896.0970	1282486.3250	1785.4460	1	11	324934.9780	1282017.8120	1790.0180	3
2	328970.8920	1282698.9600	1850.3570	2	12	324935.8090	1280237.1920	1788.8950	3
3	320440.2280	1281008.2890	1809.9550	2	13	322323.0640	1280936.9940	1792.6660	3
4	321913.1960	1280431.7470	1798.0460	2	14	323718.6360	1281270.4880	1790.3220	3
5	324137.7670	1282169.5200	1788.0050	2	15	326876.2820	1282948.1150	1802.8980	3
6	325704.1700	1281233.9510	1789.0490	2	16	321460.2750	1283477.9450	1790.3010	3
7	323299.1440	1282633.3770	1789.9280	2	17	323866.3250	1279008.5680	1787.7910	3
8	324150.3470	1280435.8530	1788.2270	2	18	322132.8520	1282903.2540	1793.9870	3
9	321352.9940	1282424.3060	1795.2320	2	19	319593.0610	1282383.5290	1810.0910	4
10	317792.1700	1279508.5920	1837.2070	2	20	323038.1560	1280528.5830	1788.7860	4

Table 2

b. The static GNSS coordinate results in meter and position and height quality (P + HQ) in millimeter (mm) obtained from LGO software for Harar city.

ID	E (m)	N(m)	H (m)	P+ HQ	ID	E (m)	N(m)	H (m)	P+ HQ
1	183158.6980	1030068.8720	1945.2170	3	11	181901.2320	1029795.4090	2005.2370	4
2	185019.7080	1029974.9170	1816.7620	5	12	183822.1180	1031012.5060	1904.9140	4
3	184358.3940	1029817.7830	1853.6390	1	13	183660.7830	1030189.3320	1896.5070	4
4	185146.7060	1029366.5040	1816.6670	6	14	183701.8670	1029773.9230	1892.6880	3
5	185093.7480	1028859.4550	1848.5080	4	15	182164.3720	1030485.8740	1991.2980	3
6	182623.8040	1030879.4270	1962.3720	6	16	182429.6870	1030442.5720	1974.4500	3
7	183655.6680	1030630.6260	1899.2370	6	17	184521.7780	1030386.0060	1877.1880	7
8	183062.8410	1031132.0410	1934.7650	4	18	185271.0780	1030269.3480	1846.8310	4
9	184722.6910	1030831.2540	1872.1070	4	19	185895.3770	1030213.6350	1817.3380	4
10	181894.8580	1029564.4680	1993.6570	3	20	185964.9640	1029990.4480	1798.0570	3

Table 2

c. The static GNSS coordinate results in meter and position and height quality (P + HQ) in millimeter (mm) obtained from LGO software for Debre Markos city.

ID	E (m)	N(m)	H (m)	P+ HQ	ID	E (m)	N(m)	H (m)	P+ HQ
1	362289.6690	1138878.3330	2392.6810	4	11	360729.4000	1142585.6210	2456.8630	2
2	361854.2550	1139238.7210	2385.2050	4	12	360839.4420	1143306.5420	2467.0810	4
3	360469.8120	1144622.2460	2432.7540	5	13	360839.2230	1143837.6260	2478.9370	5
4	361764.5840	1140517.3070	2397.9140	6	14	359027.1990	1144618.2280	2376.3800	6
5	361021.1600	1141573.8130	2401.9940	1	15	361628.8650	1143579.0700	2430.1620	3
6	360520.8400	1141915.4060	2473.8760	3	16	361415.1100	1141899.9700	2395.0390	2
7	360815.4350	1142131.6300	2452.2430	4	17	359996.5520	1143819.2260	2421.3170	4
8	360200.2990	1140672.1470	2480.7390	5	18	361602.6430	1142974.6900	2414.4650	5
9	360418.8210	1142782.4920	2436.9520	7	19	361007.5750	1144697.2970	2466.0600	4
10	361442.1400	1142572.1070	2407.6820	3	20	360078.7390	1143235.6530	2400.5730	8

In presenting the results, the study used the GNSS results using the LGO as true values (see Table 2a, 2b and 2c). Agreements between the GNSS result and orthophoto/DEM derived coordinates were computed across three different experimental sites under three different scenarios. To streamline the

presentation of tabular results, only the difference in easting and northing were computed and presented (See Tables 3, 4 and 5). For all cities and scenarios, height values were extracted from the DEM which is the input/prior output of the given orthophoto. Deviations between DEM-derived and GNSS-derived heights were computed in terms of mean error, standard deviation and RMSE. The heights values from DEM were extracted using GIS platform specifically Arc tool box, 'Extracting Values to Point' function. In doing so, the input point features are candidate CPs and the input raster is DEM. As a result, the output features are point feature dataset containing the extracted raster values.

5.2 Scenario Based Result

5.2.1 First Scenario Positional Point Accuracy

The present study relies on mean error, standard deviation, RMSE, and the 95% confidence interval for positional accuracy assessment (see Tables 3, 4, and 5). The national error budget standard of Ethiopia explicitly specifies that coordinate disagreement between geospatial datasets and reference ground data should be expressed in terms of RMSE. Accordingly, RMSE was used as the primary metric across all scenarios and experimental cities to quantify coordinate disagreement between the two datasets and the sensitivity analysis also relied on it. RMSE is considered the most suitable overall indicator because it represents the true magnitude of positional disagreement and is a standardized, widely accepted measure endorsed by ISO, ASPRS, and NSSDA/FGDC.

In the first scenario, the accuracy of horizontal coordinates of the CPs derived from orthophoto was evaluated by comparing it with the corresponding coordinates measured directly from in-situ GNSS data. The agreement between orthophoto derived coordinates with GNSS static observed coordinates at ten (10) checkpoints locations in easting and northing produced a root mean square errors (RMSEr) of $\pm 37\text{cm}$, $\pm 41\text{cm}$ and $\pm 32\text{cm}$ for Bahir Dar, Harar and Debre Markos city, respectively. Similarly, the vertical accuracy was assessed at the same checkpoints locations, and resulted in RMSEr of 112cm, 78cm and 66cm for Bahir Dar, Harar and Debre Markos cities, respectively. The results indicate that, all results are within the specified national error budget across three experimental sites using 10 sampling checkpoints for the horizontal component.

Table 3
Comparison between GNSS coordinates and orthophoto/DEM derived coordinates in Adindan UTM, units: meter, centimeter.

ID	Bahir Dar			Harar			Debre Markos		
	ΔE (m)	ΔN (m)	ΔH (m)	ΔE (m)	ΔN (m)	ΔH (m)	ΔE (m)	ΔN (m)	ΔH (m)
1	-0.286	0.238	-0.700	0.032	-0.317	0.672	-0.075	0.220	1.101
2	-0.295	-0.285	-1.618	-0.265	0.141	0.816	-0.162	-0.005	0.657
3	-0.335	0.013	-1.297	-0.349	0.175	1.136	-0.414	-0.018	0.395
4	-0.384	-0.084	-1.509	-0.501	0.024	0.947	0.168	0.398	0.417
5	-0.333	0.172	-0.431	-0.325	0.362	0.521	-0.254	0.282	0.513
6	-0.236	0.025	0.596	-0.171	-0.014	1.124	-0.251	0.315	0.281
7	0.201	0.137	1.298	0.136	-0.104	0.474	-0.330	0.161	0.012
8	-0.394	0.159	1.682	0.557	-0.521	0.356	-0.095	0.292	1.104
9	-0.216	-0.356	-0.328	0.039	0.235	-0.581	-0.003	0.167	0.355
10	-0.288	0.276	-0.629	-0.316	-0.214	0.635	0.228	-0.027	0.750
N ^o . of CPs	10								
Sum	-2.564	0.296	-2.937	-1.162	-0.233	6.101	-1.189	1.783	5.582
Mean error (m)	-0.256	0.030	-0.294	-0.116	-0.023	0.610	-0.119	0.178	0.558
Standard dev. (m)	0.171	0.214	1.143	0.312	0.271	0.496	0.207	0.152	0.350
RMSE _{xy} (m)	0.303	0.206	1.124	0.318	0.258	0.770	0.230	0.229	0.650
RMSE _r (m)	± 0.366			± 0.409			± 0.325		
95% CL (ASPRS, STANAG2215, FGDC and FEMA)	0.63		2.195	0.71		1.509	0.56		1.274

5.2.2 Second Scenario Positional Point Accuracy

The agreement of orthophoto derived coordinates with GNSS static observed coordinates at fifteen (15) checkpoints location in easting and northing resulted in a root mean square error (RMSEr) of $\pm 38\text{cm}$, $\pm 40\text{cm}$ and $\pm 33\text{cm}$ for Bahir Dar, Harar and Debre Markos city respectively. Likewise, the vertical accuracy at the same checkpoints location resulted in a root mean square error of 96cm, 94cm and 71cm for Bahir Dar, Harar and Debre Markos cities, respectively. The results indicate that, all results are within the specified national error budget across three experimental sites using 15 sampling checkpoints for the horizontal component.

Table 4
Comparison between GNSS coordinates and orthophoto/DEM derived coordinates in Adindan UTM, units: meter, centimeter.

ID	Bahir Dar			Harar			Debre Markos		
	ΔE (m)	ΔN (m)	ΔH (m)	ΔE (m)	ΔN (m)	ΔH (m)	ΔE (m)	ΔN (m)	ΔH (m)
1	-0.286	0.238	-0.700	0.032	-0.317	0.672	0.227	0.173	0.902
2	-0.295	-0.285	-1.618	-0.425	-0.181	0.114	-0.075	0.220	1.101
3	-0.295	-0.110	-1.138	-0.235	0.019	0.597	-0.162	-0.005	0.657
4	-0.103	-0.337	-1.350	0.096	0.261	1.648	-0.414	-0.018	0.395
5	-0.384	-0.084	-1.509	-0.501	0.024	0.947	-0.140	0.429	0.304
6	-0.443	-0.575	-0.822	-0.619	-0.273	0.619	0.168	0.398	0.417
7	-0.333	0.172	-0.431	-0.325	0.362	0.521	-0.106	-0.034	0.705
8	-0.226	-0.247	-0.684	-0.426	0.102	1.250	-0.218	0.386	0.485
9	-0.145	0.176	-0.359	0.136	-0.104	0.474	-0.061	0.047	0.297
10	0.201	0.137	1.298	0.027	-0.139	0.840	-0.438	-0.331	0.290
11	-0.210	-0.018	1.107	0.357	-0.521	0.356	-0.251	0.315	0.281
12	0.212	0.322	0.721	0.039	0.235	-0.581	-0.090	0.316	1.267
13	0.013	-0.299	0.062	-0.168	-0.137	-1.441	-0.173	0.099	0.828
14	-0.216	-0.356	-0.328	-0.316	-0.214	0.635	-0.095	0.292	1.104
15	-0.288	0.276	-0.629	-0.484	-0.021	1.720	-0.003	0.167	0.355
N ^o . of CPs	15								
Sum	-2.797	-0.989	-6.381	-2.813	-0.903	8.372	-1.831	2.452	9.387
Mean error (m)	-0.186	-0.066	-0.425	-0.188	-0.060	0.558	-0.122	0.163	0.626
Standard dev. (m)	0.195	0.277	0.894	0.287	0.237	0.793	0.178	0.208	0.341
RMSE _{x,y} (m)	0.265	0.275	0.963	0.332	0.230	0.948	0.211	0.259	0.707
RMSEr (m)	± 0.382			± 0.404			± 0.334		
95% CL (ASPRS, STANAG2215, FGDC and FEMA)	0.66		1.887	0.70		1.858	0.57		1.385

5.2.3 Third Scenario Positional Point Accuracy

The agreement of orthophoto-derived coordinates with GNSS static observation coordinates at twenty (20) checkpoints location in easting and northing resulted in a Root Mean Square Error (RMSEr) of $\pm 39\text{cm}$, $\pm 40\text{cm}$ and $\pm 36\text{cm}$ for Bahir Dar, Harar and Debre Markos city respectively. The vertical accuracy, at the same checkpoints location, produced a root mean square error of 102cm, 93cm and 69cm for Bahir Dar, Harar and Debre Markos cities, respectively. The result indicates that all differences are within the specified national error budget across the three experimental sites using 20 sampling checkpoints for the horizontal component.

Table 5
Comparison between GNSS coordinates and orthophoto/DEM derived coordinates in Adindan UTM, units: meter, centimeter.

ID	Bahir Dar			Harar			Debre Markos		
	ΔE (m)	ΔN (m)	ΔH (m)	ΔE (m)	ΔN (m)	ΔH (m)	ΔE (m)	ΔN (m)	ΔH (m)
1	-0.286	0.238	-0.700	0.032	-0.317	0.672	0.227	0.173	0.902
2	-0.295	-0.285	-1.618	-0.122	0.452	0.468	-0.075	0.220	1.101
3	-0.295	-0.110	-1.138	-0.425	-0.181	0.114	-0.162	-0.005	0.657
4	-0.103	-0.337	-1.350	-0.235	0.019	0.597	-0.414	-0.018	0.395
5	-0.335	0.013	-1.297	-0.265	0.141	0.816	-0.140	0.429	0.304
6	-0.285	-0.076	-1.261	-0.349	0.175	1.136	-0.378	0.165	1.091
7	-0.384	-0.084	-1.509	-0.295	-0.124	0.722	0.168	0.398	0.417
8	-0.443	-0.575	-0.822	0.096	0.161	1.648	-0.106	-0.034	0.705
9	-0.439	-0.526	-0.770	-0.501	0.024	0.947	-0.218	0.386	0.485
10	-0.333	0.172	-0.431	-0.619	-0.273	0.619	-0.254	0.282	0.513
11	-0.226	-0.247	-0.684	-0.325	0.362	0.521	-0.295	0.545	0.209
12	-0.145	0.176	-0.359	-0.426	0.102	1.250	-0.061	0.047	0.297
13	-0.236	0.025	0.596	-0.171	-0.014	1.124	-0.438	-0.331	0.290
14	0.201	0.137	1.298	0.136	-0.104	0.474	-0.251	0.315	0.281
15	-0.210	-0.018	1.107	0.027	-0.139	0.840	-0.090	0.316	1.267
16	-0.394	0.159	1.682	0.557	-0.521	0.356	-0.173	0.099	0.828
17	0.212	0.322	0.721	0.039	0.235	-0.581	-0.330	0.161	0.012
18	0.013	-0.299	0.062	-0.168	-0.137	-1.441	-0.095	0.292	1.104
19	-0.216	-0.356	-0.328	-0.316	-0.214	0.635	-0.003	0.167	0.355
20	-0.288	0.276	-0.629	-0.484	-0.021	1.720	0.228	-0.027	0.750
N°. of CPs	20								
Sum	-4.485	-1.394	-7.430	-3.815	-0.373	12.638	-2.861	3.576	11.961
Mean error (m)	-0.224	-0.070	-0.372	-0.191	-0.019	0.632	-0.143	0.179	0.598
Standard dev. (m)	0.184	0.266	0.978	0.277	0.236	0.706	0.194	0.205	0.356
RMSE _{x,y} (m)	0.287	0.269	1.024	0.330	0.231	0.934	0.237	0.268	0.691
RMSEr (m)	± 0.393			± 0.403			± 0.358		
95% CL (ASPRS, STANAG2215, FGDC and FEMA)	0.68		2.007	0.70		1.830	0.62		1.354

The overall horizontal positional result of this study computed in terms of RMSE in easting and northing, were $\pm 36\text{cm}$, $\pm 40\text{cm}$, and $\pm 32\text{cm}$; $\pm 38\text{cm}$, $\pm 40\text{cm}$, and $\pm 33\text{cm}$; and ± 39 , ± 40 and $\pm 35\text{cm}$ when using 10, 15 and 20 sampling checkpoints in Bahir Dar, Harar and Debre Markos cities, respectively. Whereas, the vertical positional result were $\pm 112\text{cm}$, $\pm 78\text{cm}$, and $\pm 66\text{cm}$; $\pm 96\text{cm}$, $\pm 94\text{cm}$, and $\pm 71\text{cm}$; and ± 102 , ± 93 and $\pm 69\text{cm}$ when using 10, 15 and 20 sampling checkpoints in Bahir Dar, Harar and Debre Markos cities, respectively.

The relatively high vertical positional error can be attributed to two main factors. First, artificial surface features were not adequately removed during DEM generation, as the primary objective of the orthophoto production was two-dimensional mapping applications rather than three-dimensional surface modeling. Second, Ethiopia lacks a globally published, country-specific vertical datum; consequently, orthometric heights are commonly derived from GNSS data using global geoid models such as the Earth Gravitational Model 1996 (EGM96) or the Earth Gravitational Model 2008 (EGM2008). Therefore, artificial surface objects mainly introduce local, random vertical errors, while the use of global geoid models may result in systematic height offsets due to incomplete representation of local geoid undulations and potential regional biases in GNSS-derived orthometric heights.

6. Discussions and Conclusions

Currently, many geospatial sectors employ an integrated surveying approach (ground-based, aerial and high resolution satellite products) for applications such as; land registration urban and rural planning, irrigation site development, and any other spatial oriented projects (MoUI, 2015). This paper applies

national and international standards, together with scientific methods of photogrammetric science, to evaluate positional point accuracy in urban orthophoto mapping. Three cities, with varied topographic characteristics, Bahir Dar, Harar and Debre Markos, were selected as test case, to examine positional accuracy under multiple experimental scenarios using sensitive analysis and multi-standards frameworks for validating point location accuracies achieved in orthophoto mapping. A set of carefully selected check point were measured using static GNSS and processed using LGO software package. The present study extends conventional positional accuracy assessment by explicitly incorporating sensitivity testing and a multi-standard evaluation approach, allowing a more comprehensive interpretation of positional accuracy behavior under varying sampling CPs scenarios. The RMSE values obtained for easting and northing together demonstrate that positional accuracy remains stable when the number of sampling CPs increases from 10 to 20 within the same city. The sensitivity testing revealed a maximum positional accuracy difference of 3cm and a minimum difference of 0.5cm within individual cities, indicating that the applied methodology is not highly sensitive to changes in sampling CP density. This confirms that the positional accuracy results are robust and repeatable under different sampling scenarios.

The multi-standard approach further strengthens the reliability of the findings by ensuring that positional accuracy is not evaluated under a single accuracy framework. While previous studies often relied on a single standard or fixed CP configuration, this study demonstrates that consistent RMSE behavior can be achieved across multiple scenarios, supporting the applicability of the results under different accuracy evaluation standards. The inter-city sensitivity testing results, which show a maximum point positional accuracy difference of 8.9cm and a minimum of 0.9cm across Bahir Dar, Harar, and Debre Markos cities, emphasize that contextual factors related to geographic location influence positional accuracy more than the number of sampling CPs. These findings are in agreement with previous related research that reported high positional accuracy using standardized assessment methods and appropriate checkpoint strategies. Studies by(Drobnjak & Bozic, 2018; Mesas-Carrascosa et al., 2014; Radojčić, 2022) demonstrated that acceptable positional accuracy can be achieved when established standards such as(FEMA, 2003; NMAS, 1947; NSSDA/FGDC, 1998; NSSDA/FGDC, 2002; STANAG 2215, (2010)) are applied. However, unlike these studies, which primarily focused on single-scenario assessments, the present study explicitly evaluates the sensitivity of RMSE to varying CP numbers, providing additional insight into the stability of positional accuracy results.

More recent UAV-based studies, includingAtik et al. (2025), reported decimeter- to centimeter-level accuracy, reflecting advances in data acquisition and processing techniques. The RMSE values obtained in the present study are comparable with these findings, particularly when considering the multi-standard evaluation framework and sensitivity testing approach employed. This comparison highlights that the integration of sensitivity testing with multi-standard assessment enhances confidence in positional accuracy results by demonstrating consistency across different evaluation conditions. Furthermore,Thabani T. et.al.(2024) compared the UAV coordinates with total station coordinates and reported root mean square errors (RMSE) of $\pm 0.046\text{m}$, $\pm 0.038\text{m}$, and $\pm 0.079\text{m}$ for the X, Y, and H coordinates, respectively, at 159 CP locations. Overall, combining sensitivity testing with a multi-standard positional accuracy assessment provides a more rigorous evaluation framework than single-scenario approaches. The minimal RMSE variation within the same city and the moderate variation across different cities demonstrate that the positional accuracy outcomes are both methodologically stable and contextually influenced. This reinforces the importance of incorporating sensitivity analysis and multiple accuracy standards in geospatial positional accuracy assessment to ensure reliable and transferable results.

Table 6
Summary of RMSEr difference across cities with the same scenario and RMSEr difference cross scenario with the same city

Scenarios	Cities with its RMSEr			Differences across cities, but the same scenario		Differences across scenario, the same city	
	Bahir Dar	Harar	Debre Markos	Differences Max.	Differences Min.	Differences Max.	Differences Min.
1 (10 CPs)	36 cm	40.9 cm	32 cm	8.9 cm	4 cm	3 cm	2 cm
2 (15 CPs)	38 cm	40.4 cm	33 cm	7.4 cm	2.4 cm	0.6 cm	0.5 cm
3 (20 CPs)	39 cm	40.3 cm	35 cm	5.3 cm	1.3 cm	3 cm	1 cm

For instances, in Debre Markos city specifically in scenario 1 the RMSEr is 32cm, which is minimum, whereas, in Harar city the RMSEr 40.9cm was observed as shown in Table 6 and Fig. 5. The resulting difference of 8.9cm indicates that the maximum variation in point positional accuracy occurs between cities having varied topographic characteristics, rather than within the same city under different sample sizes.

All the positional accuracy results obtained across the three scenarios and the three experimental sites, generally meet the requirement set in Sub-section 2.3, of the Ethiopia's Urban Legal Cadastral Standard No.-03/2015, which sets a maximum allowable error budget of, $\pm 40\text{cm}$ horizontally for a map scale 1:2,000 in urban areas. The coordinate disagreement between Orthohpto and GNSS static derived in easting, northing and height at each sampling locations exhibit a systematic shift, as seen above in Tables 3, 4 and 5. In this context, the numbers of checkpoints aren't as such the decisive factor to acquire the optimum results. However, the RMSE tend to decrease when the number of CP increased from 10 to 20 across all scenarios and cities as shown in Fig. 6. In contrary, appropriate location such as sharp or visible corners of manmade features and careful consideration of topographic variations within each experimental site play critical role in improving positional accuracy.

In this case of sensitivity testing, it can be concluded that, across all scenarios within the same city, the maximum point positional accuracy difference is 3cm and the minimum is 0.5 cm. In contrast, across the three cities and all scenarios, the maximum point positional accuracy difference is 8.9cm, while the minimum is 0.9cm expressed in terms of combined easting and northing RMSEr, as shown in the Table 6. As demonstrated in Table 3 (first scenario), Table 4 (second scenario) and Table 5 (third scenario), positional accuracy results do not vary significantly, they are not highly sensitive when the numbers and spatial distribution of CPs are varied.

Declarations

Conflict of Interest

There is no conflict of interest regarding the publication of this research.

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Author Contribution

The first author (Zinabu Getahun Sisay, Assistant Professor) led the study, collected and analyzed data, and drafted the manuscript. The corresponding author (Solomon Dargie Chekole, PhD) oversaw the research, guided methodology, analysis, and managed submission. The third author (Wubante Fetene Admasu, PhD)) supported design, analysis, and manuscript review. All authors approved the final manuscript.

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Data Availability

The GNSS static DBX files measured and analyzed in this study have been uploaded to the submission system as “Supplementary file” and are also available upon reasonable request. Sample photographs/pictures from the GNSS data collection are likewise provided as a “Supplementary file”. Other datasets, including digital orthophotos and photogrammetrically derived Digital Elevation Models (DEMs) obtained from the Ministry of Urban and Infrastructure, are not publicly available due to access restrictions, but may be obtained from the authors upon reasonable request and subject to permission from the data owner

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Figures

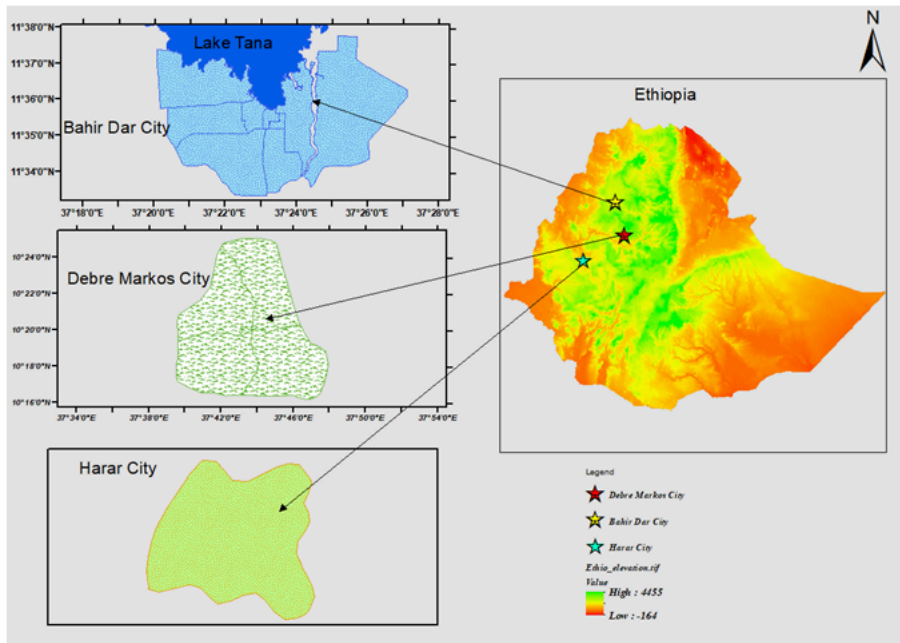


Fig. 1 Study Area Map

Figure 1

Study Area Map

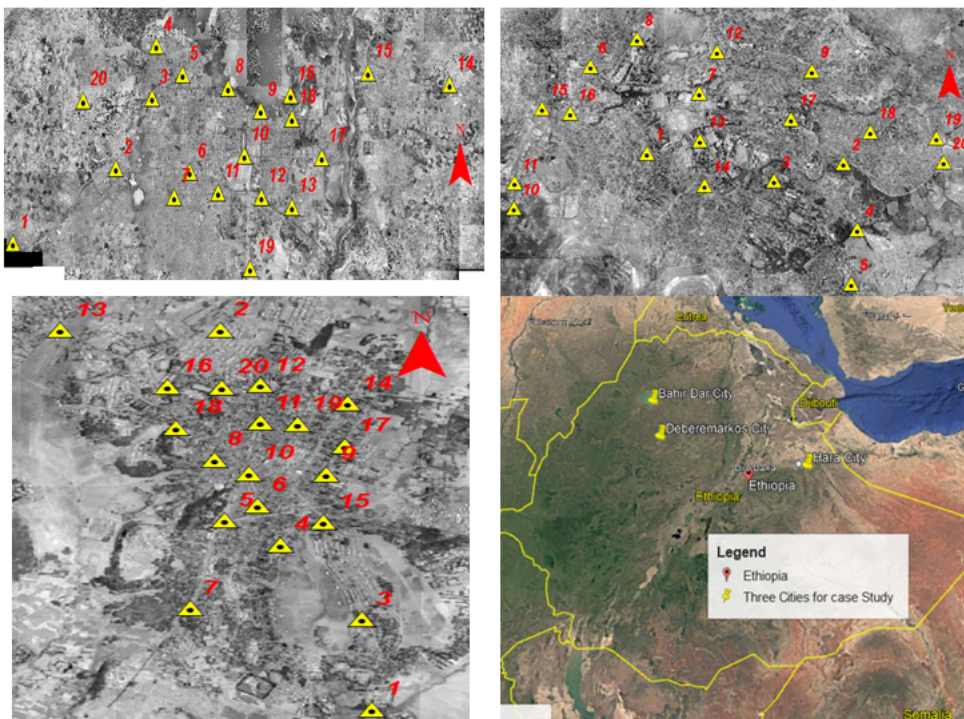


Fig. 2 Checkpoint location symbolized by yellow triangles in 1st, Bahir Dar, 2nd, Debre Markos and 3rd Harar at Canal corners, Roads junctions, water point, Culvert edge, Centers of utilities, Bridge corner, Swimming pool edge

Figure 2

Checkpoint location symbolized by yellow triangles in 1st, Bahir Dar, 2nd, Debre Markos and 3rd Harar at Canal corners, Roads junctions, water point, Culvert edge, Centers of utilities, Bridge corner, Swimming pool edge



Fig.3 GNSS Measurement Sample Location

Figure 3

GNSS Measurement Sample Picture

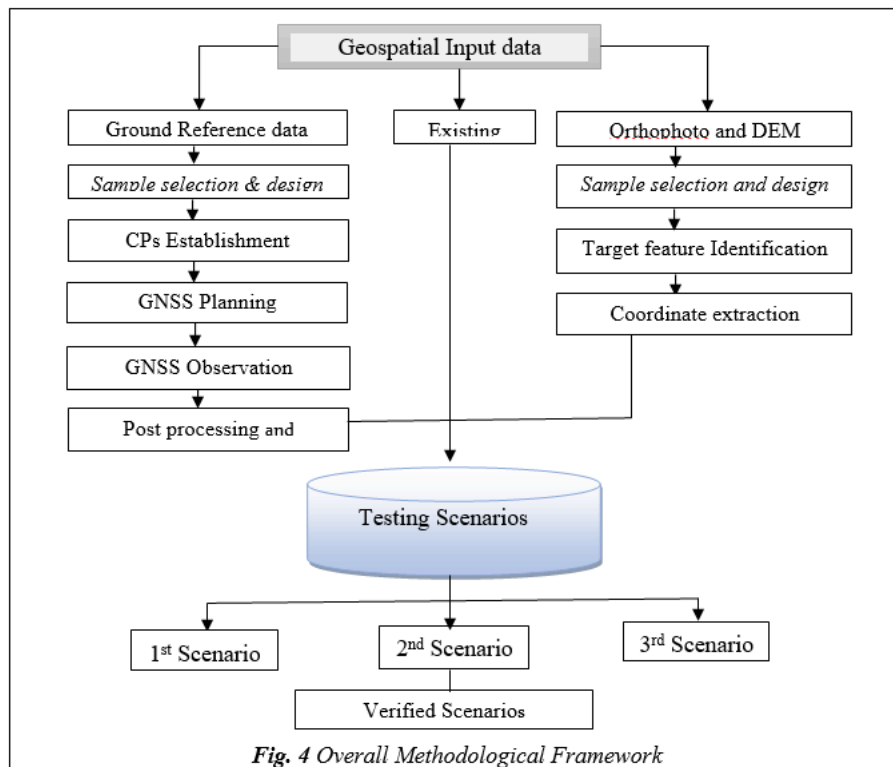


Fig. 4 Overall Methodological Framework

Figure 4

Overall Methodological Framework

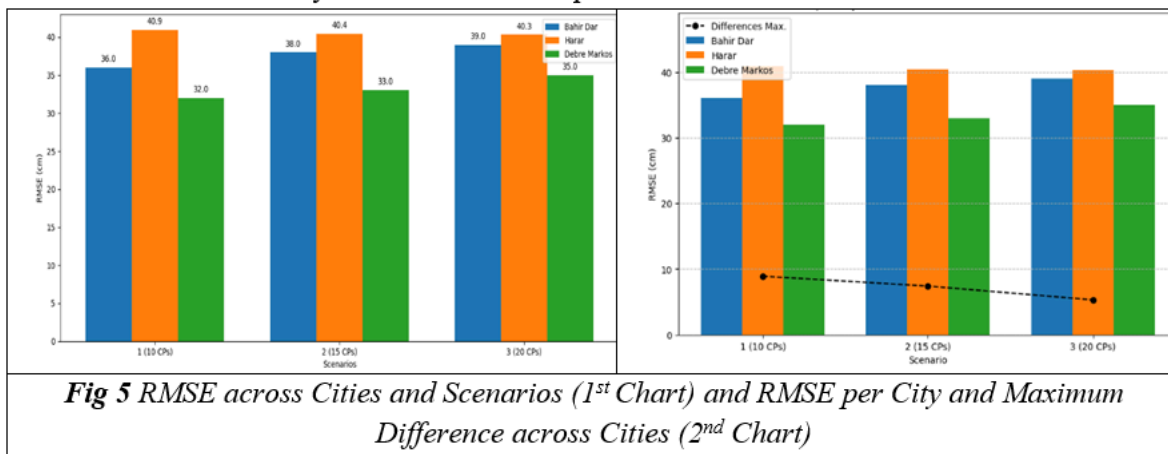


Figure 5

RMSE across Cities and Scenarios (1st Chart) and RMSE per City and Maximum Difference across Cities (2nd Chart)

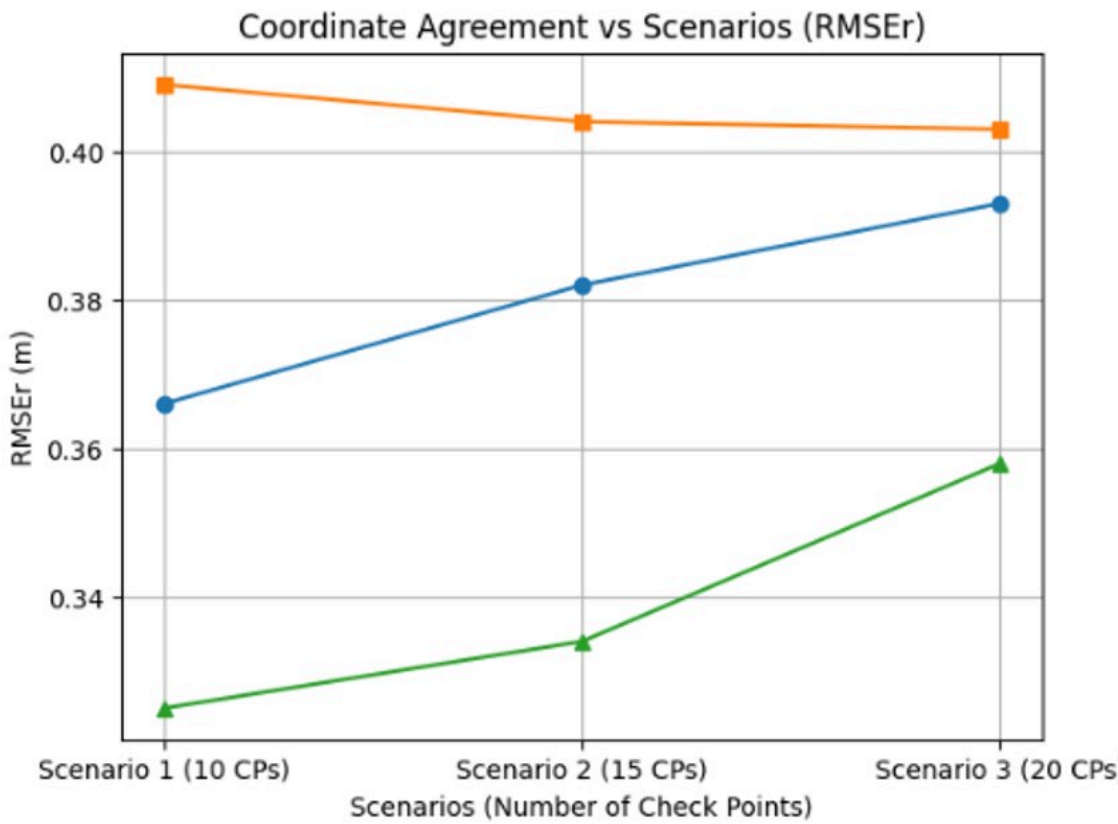


Figure 6

Coordinate disagreements across the experimental cities against the RMSE referring that the red color for Harar City, the blue color for Bahir Dar, and the green color for Debre Markos city

Supplementary Files

This is a list of supplementary files associated with this preprint. Click to download.

- [GNSSRawDataCollectionSamplePicture.rar](#)
- [GNSSRawData.rar](#)