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Positional Accuracy Validation of Digital Orthophoto Mapping: Case Bahir Dar City, Ethiopia

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Abstract. *This study used in-situ Global Positioning System (GPS) measurements to assess the accuracy of horizontal coordinates of the orthophoto map for Bahir Dar city. The GPS data was least-squares adjusted using the GAMIT/GLOBK and Leica GeoOffice (LGO) software packages. Local and regional GPS reference stations, including the continuously operating reference station of Bahir Dar University's Institute of Land Administration, were included in the adjustment. Thus, horizontal coordinates at five check-points were obtained, which were used to assess the horizontal positional accuracy of these same points in the orthophoto map.*

Point accuracies found for point locations read from the orthophoto map were inferred to be on the level ± 0.3 m. This meets the requirement of the Ethiopian Mapping Authority of ± 0.30 m for maps on scale 1:2,000, which are sufficient for cadastral and land-use planning use everywhere, also in urban areas, though not perhaps in dense city centres.

Key words. *Ethiopia, urban planning, orthophoto map, Global Positioning System, positional accuracy, cadastre*

1 Introduction

According to Hailu and Harris (2014), there are more than 50 million rural land parcels in Ethiopia belonging to some 26 million land holders. The country is carrying out an ambitious plan to consult all these land holders with a view to reliably and fairly recording and mapping parcel boundaries, using orthophoto mapping technology by aerial or satellite imagery. The mapping will serve not only the cadastral system, but also both rural and urban sustainable land-use systems.

The focus of the current paper, however, is horizontal point location accuracy in urban orthophoto mapping. The accuracy demands for urban mapping are higher than for general orthophoto mapping, and its primary focus is urban land-use planning rather than just the registration of parcel boundaries. Also, the challenges of mapping urban landscapes precisely are special, e.g., Zhou and Chen (2002); Zhou et al. (2004); Palà and Arbiol (2002). This is an issue of global importance as many newly industrializing countries are characterized by rapid, often uncontrolled, urbanization; e.g., Chhatkuli (2002); Akdeniz (2004).

However, so far, no positional accuracy assessment of orthophoto mapping has been performed over Ethiopia. The intention of this paper is to study the accuracy achievable for terrain points using the city of Bahir Dar, the capital of Amhara National Regional State, as a real-life test case. In particular, the study evaluates how well do orthophoto based horizontal coordinates of surface points agree with static GPS measurements. In this regard, the paper also investigates the consistence between GPS derived coordinates acquired from different processing software packages like GAMIT/GLOBK, Leica GeoOffice, AUSPOS and APPS. As such, it is a validation of mapping technologies and methodologies used in a typical, medium-sized Ethiopian city. Bahir Dar is a city in North-Western Ethiopia at the Southern shore of Lake Tana, ≈ 567 km (ground distance) NNW of Addis Ababa, at an elevation of 1,800 m. It is one of the fastest expanding towns in Ethiopia and, with a beautifully laid out inner city and close to the waterfalls of the Blue Nile, sports a successful tourist industry. The city has a quarter million inhabitants and growing, up from 50,000 inhabitants in 1984.

2 Materials and methods

2.1 Data used

This study used, as accuracy assessment sample data, an orthophoto and digital line map acquired from aerial photogrammetric survey carried out over the whole area of Bahir Dar Town, accounting for a total area of 159 km². The photogrammetric surveying was conducted by the Information Network Security Agency (INSA, <http://www.insa.gov.et/>) in 2011/12 with a standard aerial camera at 1:2,000 scale and 15 cm Ground Sample Distance (GSD). The orthophoto and digital line map was obtained from the Amhara National Regional State Office of Urban Development and Industry. Originally, the orthorectification process was performed by the Ethiopian Mapping Agency and INSA using ArcInfo software, while the extraction of the digital line map was carried out using PromtMap software by GeoMarc Systems PLC (<http://www.geomark.com.et/>). Besides, this study used processed ground based static GNSS data collected at spatial locations selected from this orthophoto and digital line map.

2.2 Point selection and accuracy criteria

2.2.1 Widely used standards

This study is guided by two newly endorsed test standards: ASPRS (2013) for designing the distribution of checkpoints and FGDC (1998) for determining the sample size. Besides, this study attempts to examine the accuracy assessment from the perspective of Ethiopian standards and directives, e.g., Ministry of Urban Development, Housing and Construction (2015).

Estimating errors in the positional co-ordinates of spatial objects requires the comparison of co-ordinates (x, y) of identical sample locations from photogrammetric data with those of the real objects on the surface of the Earth. The *cluster sampling technique* is used due to the fact that selection and distribution of

the checkpoints (CP's) primarily depend on visibility and recoverability of well-defined points on an orthophoto by ground observation using a GNSS receiver. Besides, the samples shall be representative in view of topographic and land-use variation. The appropriate locations of CP's could be canal corners, roads junctions, centers of utilities, swimming pool edges, etc.

The selection of check points and their spatial distribution was guided by the 1998 international standard of the U.S. National Standard for Spatial Data Accuracy (NSSDA) authored by the U.S. Federal Geographic Data Committee (FGDC). The FGDC (1998) standard¹ provides excellent guidance on positional accuracy sample design, estimation of sample size and sample selection criteria. By this standard, we have a number of basic requirements for sampling design and selection. Of course, the independence of the reference data from the data being tested is of critical importance. Then, there are four criteria:

1. The reference data should be at least three times more accurate than the data being assessed.
2. A minimum of 20 sample points are required to test positional accuracy with reliable statistical rigor. The present study falls short on this criterion.
3. The selection of the samples must consist of "well-defined points" that are easily visible or recoverable on the ground.
4. Sample points should be well distributed across the experimental site, and represent the full variety of topography.

The standard ASPRS (2013) also explains that the selection of the number of checkpoints depends on the area of the experimental site. For instance, study sites having areas of ≤ 500 , 501 – 750, 751 – 1,000 square kilometers are required to have 20, 25, and 30 checkpoints, respectively. The standard FGDC (1998) recommends that systematic sampling methods be used to ensure that sample points are well distributed. A minimum of 20% of the sample points are allocated to each quadrant, no two points should be closer than $\frac{d}{10}$, where d is the diagonal dimension of the map or image².

2.2.2 Other standards

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

One should be aware that the accuracy requirement, just like the choice of mapping scale, will be directly related with the *intensity of land use*. For this reason, some countries, like Finland, use a classification of land into measurement

¹ The European Union standard Kapnias et al. (2008) appears somewhat similar.

² It is easily derived that this limits the maximum total number of sample points to $n \leq 50$.

classes, which are defined by the monetary value of the land, and in turn defines the scale at which the land needs to be mapped, and the accuracy on the ground that the map should meet. See the summary in Table 1. Obviously, the Ethiopian situation differs from the Finnish one, so the concrete numbers cannot be copied.

Table 1. Measurement classes, according to the Finnish standard JUHTA (2014). One should appreciate that in Finland, lake or sea shore (measurement class 3) is coveted for its recreational use.

<p>Survey areas are divided into three measurement classes. The measurement class defines the accuracy of measurement and graphical rendering.</p> <p>Measurement class 1: Urban areas where land is very valuable and where there is a valid local zoning plan with a binding parcel division, or a construction ban awaiting the drafting of such a plan.</p> <p>In surveys intended to be incorporated into the municipal GIS for use in technical planning requiring great accuracy, a higher accuracy level may be used (<i>measurement class 1e</i>).</p> <p>Map scale 1:500 or 1:1,000. Parcel boundary co-ordinate accuracy (mean error or standard deviation) ± 0.085 m (point uncertainty ± 0.12 m).</p> <p>Measurement class 2: Urban areas for which the local zoning plan to be drafted does not require binding parcel division.</p> <p>Map scale 1:1000 or 1:2000. Parcel boundary co-ordinate accuracy ± 0.14 m (point uncertainty ± 0.20 m).</p> <p>Measurement class 3: Areas zoned as lake or sea shore, lake or sea shore areas, and other areas where the land is clearly more valuable than agricultural or forest land, e.g., so-called dispersed settlements (https://en.wikipedia.org/wiki/Dispersed_settlement).</p> <p>Map scale is usually 1:2,000. Parcel boundary co-ordinate accuracy ± 0.21 m (point uncertainty ± 0.30 m).</p> <p>A scale of 1:4,000 or 1:5,000 may be accepted if a zoning plan can be drafted with such a map without essentially compromising the requirements to be placed on it.</p> <p>Measurement class 4: All other (unzoned) areas.</p> <p>A digital map has no scale. The accuracy of the collected data corresponds to that on a map of a certain scale. One should not print out graphical products from a digital data base at a scale greater than the accuracy of the data allows.</p> <p>In choosing the measurement method and the scale of the map, one must take into account surveys carried out earlier in the area or its surroundings, and the extent and character of the area.</p>

Table 2. Measurement sessions. DoY (Day of Year) numbers for 2016.

DoY	111	112	113	114	115	116	117	118	119	120	121
GPS1	1	1	1	1							
GPS2					2	2	2	2			
GPS3	1	1	1	1							
GPS4								3	3	3	3
GPS5					2	2	2	2			

2.2.3 Sample point selection

For the present study, we intended the sample locations to be determined on the basis of the standards and criteria as discussed above. The spatial objects are identified and selected by visualizing the digital orthophoto by zooming in and out on the ArcGIS and ERDAS Imagine platforms. In connection to this, the study combines the standards FGDC (1998); ASPRS (2013) with the equations from Greenwalt and Shultz (1992) to practice horizontal positional accuracy assessment of digital orthophotos and their extracted features.

Originally, we identified about sixteen points within the test area as suitable for use in this study. Eventually, of those, *five points* were chosen to be used with good GNSS visibility and recoverability, see Figures 1 and 2.



Figure 1. Checkpoint locations in Bahir Dar based on reconnaissance. The area depicted is about $4 \times 6 \text{ km}^2$ in size.

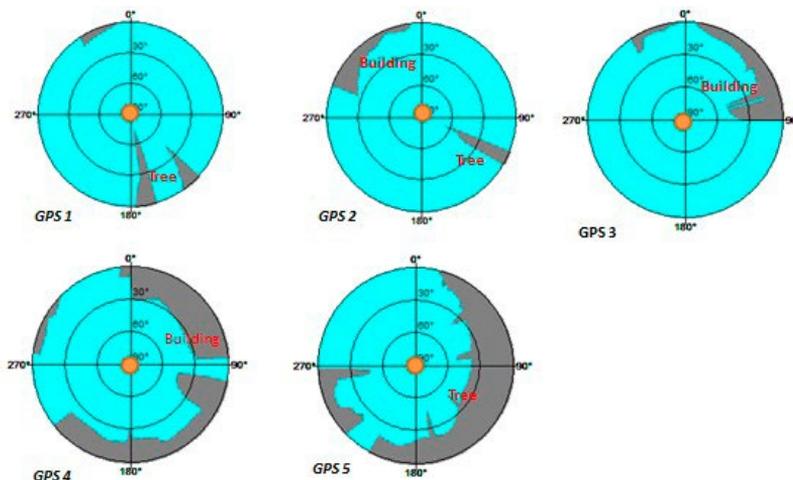


Figure 2. Obstacle sky plots of the checkpoint locations, generated by Trimble planning software.

Unfortunately this number is much smaller than that specified in item 2 in subsection 2.2.1. This was as strategy choice made early in the project, which undoubtedly would be made differently based on what we know now. For this reason, the current results must be seen as indicative only, and more extensive testing would be called for.

2.3 GNSS measurement

The network measured is depicted in Figure 3. The receiver type used was Leica GPS1200. With two instruments of this type, three static GNSS measurement sessions were carried out of duration 72 hours each, at a data logging interval of 30 seconds. During the measurements, from 8 to 10 satellites, both GPS and GLO-NASS, were in the sky, and GDOP varied between 1.2 and 2.8. The sessions occupied the points GPS1 and GPS3, GPS2 and GPS5, and GPS4, respectively.

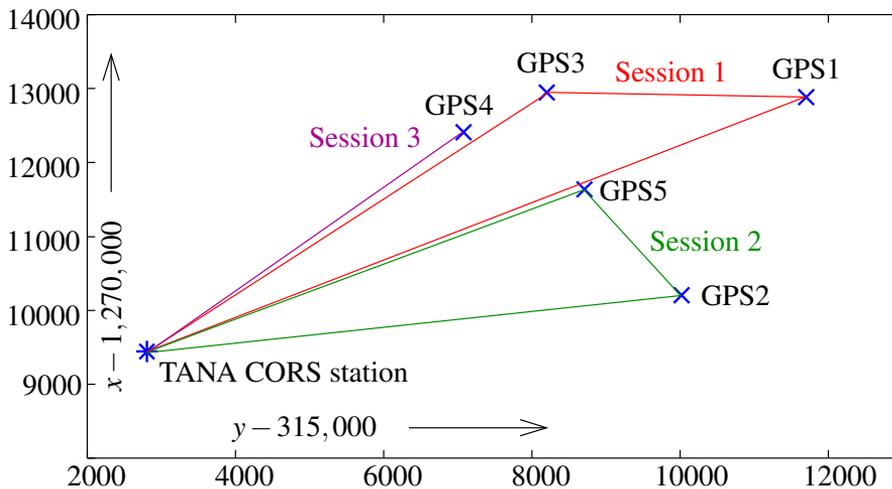


Figure 3. The GNSS network measured. Co-ordinates x, y in the Adindan UTM projection zone 37N, units: m.

2.4 GNSS processing and Continuously Operating Reference Stations (CORS)

The GNSS processing was done using two different software packages: the package GAMIT/GLOBK, originating with the Massachusetts Institute of Technology (<http://www-gpsg.mit.edu/~simon/gtgk/>) which is available free of charge, and Leica Geo-Office (LGO, <https://leica-geosystems.com/it-it/products/gnss-systems/software/leica-geo-office>) from Leica Geosystems.

Furthermore, two remote geodetic GNSS processing services were used: AUSPOS in Australia (<https://tinyurl.com/AUSPOSx>), and APPS of the NASA Jet Propulsion Laboratory (<http://apps.gdgps.net/>). AUSPOS, which is based on the Bernese software (<http://www.bernese.unibe.ch/>), allows the networked processing of observations from several user stations in RINEX format, using region-

al IGS stations for reference. APPS, which internally uses GIPSY-OASIS (<https://gipsy-oasis.jpl.nasa.gov/>), makes use of a PPP-like approach (Precise Point Positioning), where user stations are positioned one by one, again using regional IGS stations for reference. The results are very similar in practice.

In preparation for the processing, data from a number of CORS were downloaded. Firstly the station TANA, located at the Institute of Land Administration, University of Bahir Dar, was used; this station was at a distance of only 8 km from the measurement area. The receiver is a Leica GR25 with a Leica AR25 choke-ring antenna, tracking both GPS and GLONASS satellites. Secondly, the station ADIS, at the Geophysical Observatory of Addis Ababa University, was used. It is equipped with a Topcon Legacy E receiver with choke-ring antenna tracking both GPS and GLONASS satellites.

In the GAMIT/GLOBK processing, remoter IGS reference stations, like MBAR (Mbarara, Uganda), NKLK (Libreville, Gabon) and MAL2 (Malindi, Kenya) were included. All data, from the reference stations and from the measured checkpoints, was converted to the receiver independent RINEX format. While the package GAMIT/GLOBK was able to process data from all these stations, LGO only successfully processed data from the TANA CORS station, being equipped with a Leica type receiver. There appears to be an incompatibility, possibly related to receiver type, for the remoter CORS stations, most of which are equipped with different types of receivers – ADIS has a Topcon receiver, MBAR a Javad, MAL2 a Septentrio, and NKLK a Trimble...

In the processing using the AUSPOS and APPS services, also only remote IGS reference stations were used, automatically chosen by the software. TANA was not on offer. For this reason, one may expect systematic offsets in the results, due, i.a., to plate tectonics, to a different ITRS realization and epoch, and more generally to our proj4 script (algorithm 1) not precisely corresponding to what was used when the modern Adindan datum was created.

Therefore, we did a validation computation: the official co-ordinates of TANA in Adindan are known, and were compared with the AUSPOS (and APPS, and GAMIT/GLOBK) results using only remote IGS reference stations. The offsets found (Table 3) were applied to the Eastings and Northings from the GNSS computations, in order to obtain comparable co-ordinates in the official Adindan datum.

Table 3. Official and computed Adindan UTM zone 37N co-ordinates of TANA, and origin offset inferred.

	<i>E</i> (m)	<i>N</i> (m)
Official	317707.525	1279229.278
AUSPOS	317707.466	1279229.241
Offset	0.059	0.037

3 Results

3.1 GNSS processing

The results obtained with the various GNSS softwares are summarized in Table 4. It is seen that the differences between all the GNSS methods and softwares are overall sub-centimetre as expected for geodetic GNSS with long occupation times.

In describing our results, we have used the values obtained with the AUSPOS service as representative for all GNSS results. Compared to the co-ordinates read from the orthophoto map, they may be considered errorless.

Table 4. Results of the GNSS computations. Co-ordinates in Adindan UTM, units: metre, millimetre. It is seen that all differences Δ are sub-centimetre.

	AUSPOS E, N		APPS E, N		Δ (mm)	
GPS1	326693.514	1282895.829	.511	.826	-3	-3
GPS2	323211.864	1282942.532	.867	.530	3	-2
GPS3	325020.115	1280209.669	.115	.664	0	-5
GPS4	322080.538	1282414.434	.539	.433	1	-1
GPS5	323716.372	1281629.034	.369	.034	-3	0
Mean					-0.4	-2.2
Std.					2.6	1.9

	Gamit E, N		Δ (mm)		LGO E, N		Δ (mm)	
GPS1	.515	.821	1	-8	.520	.827	6	-2
GPS2	.867	.527	3	-5	.865	.529	1	-3
GPS3	.117	.661	2	-9	.124	.668	9	-1
GPS4	.541	.429	3	-5	.538	.434	0	0
GPS5	.374	.029	2	-5	.376	.028	4	-6
Mean			2.0	-6.1			4.0	-2.4
Std.			0.9	1.8			3.7	2.3

3.2 Horizontal positional accuracy assessment

We see from Table 5 that co-ordinate disagreements between GNSS and orthophoto map are on the ± 0.2 m level (one sigma). If we define the *point uncertainty* as $\sigma_p = \sqrt{\sigma_x^2 + \sigma_y^2}$, with σ_x, σ_y the co-ordinate uncertainties, we find a corresponding point uncertainty of ± 0.3 m. Note that, due to the use of only five check points (co-ordinate pairs), all these uncertainty estimates are themselves afflicted with substantial uncertainty³. For this reason, we only give a single decimal.

³ For illustration, the ten degrees of freedom (five points) χ^2 distribution yields a (5% – 95%) confidence interval of (0.225 – 0.508) m for the true value, when the estimated value is 0.293 m. By comparison, for forty degrees of freedom (twenty points) one would obtain (0.251 – 0.366) m for the same confidence limits. Always assuming normally distributed, independent data, of course.

Table 5. Point positions according to the orthophoto map, and differences of GNSS positions with it. Units: metre, millimetre. The point uncertainty is estimated as $\sqrt{166^2+242^2}$ mm = 293 mm \approx 0.3 m.

Point	Orthophoto map E, N		GNSS E, N		Δ (mm)	
GPS1	326693.797	1282895.629	.514	.829	-283	200
GPS2	323211.749	1282942.194	.864	.532	115	338
GPS3	325020.066	1280209.824	.115	.669	49	-155
GPS4	322080.565	1282414.333	.538	.434	-27	101
GPS5	323716.252	1281628.775	.372	9.034	120	259
Mean					-5	149
Std.					166	191
RMS					166	242

Algorithm 1. The cs2cs shell script call used for converting WGS84 geodetic co-ordinates to Adindan geodetic co-ordinates.

```
cs2cs +proj=latlong +datum=WGS84 +to +proj=utm
+zone=37 +ellps=clrk80
+towgs84=-162,-12,206,0,0,0,0 +units=m
-r -f "%.3f" <<EOF
<Lat Lon Height comments>
...
EOF
```

4 Discussion and conclusion

Orthophoto mapping is not a new technology, as is neither the study of its accuracy. This paper brings no scientific or engineering novelty. Although this paper uses known scientific methods of photogrammetric science, studying the horizontal point location accuracy in urban orthophoto mapping in the Ethiopian context is its novel contribution, using a technique considered for broader adoption throughout the country.

In this paper, we used the city of Bahir Dar, in North-Western Ethiopia, as a test case for validating the horizontal point location accuracies achieved in orthophoto mapping. A set of five carefully selected test points was measured using static GNSS, processed using two different software packages and two remote GNSS processing services, and used for estimating point location accuracies in the orthophoto map.

As seen above and in Table 5, horizontal point location accuracies obtained meet the requirement in sub-section 2.2.2, as described in Ministry of Urban Development, Housing and Construction (2015), of ± 0.30 m for a map scale of 1:2,000 – but only barely. It should be noted that our uncertainty estimate of ± 0.3 m is based on five points only, and is thus itself uncertain.

In Finland, the same result, with the same caveat, would only meet the requirement for parcel boundary-point accuracy for Finnish measurement class 3 points (i.e., in non-urban areas where land is clearly more valuable than general

agricultural or forest land), ± 0.30 m. With modest improvement, it could also meet the requirement for class 2 points (i.e., urban areas where land is not very valuable, point accuracy ± 0.20 m).

Outside urban areas the method used is adequate and meets ‘fitness-for-purpose’ requirements (Enemark et al., 2014). For rural and remote areas again – not the focus of this study – it might be suitably relaxed.

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