VERIFYING THE LIMITATION OF GNSS-PPP DERIVED ORTHOMETRIC HEIGHTS IN ENGINEERING DESIGN USING NON-RIGOROUS APPROACH

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Abstract

Height is a vital component in highway construction, vertical and horizontal alignments in structures, deformation monitoring, subsidence analysis and terrain model analysis. GNSS-PPP height is obtained by way of the operations of a single GNSS receiver, orbit and clock data products. The GNSS-PPP operations are conveniently carried out via the internet and e-mail services after RINEX files of observations have been sent to these various PPP platforms. This research aimed to verify the limitation of GNSS-PPP heights in engineering design using non-rigorous approach. The instruments used for this study were Automatic level and its accessories together with GNSS V30 Hi-Target receiver with its accessories. The primary data used for this research were obtained by leveling operations. The secondary data used were two GNSS-PPP coordinates which included the ellipsoidal heights and the geoidal undulation values for the two GNSS-PPP stations. The geoidal undulation values of EGM 2008 were obtained using GeoidEval utility software. The geoid eval computes the geoidal height using interpolation in grid of values. The root mean square (RMS) obtained after every interpolation was within the acceptable range of 1mm. The geographical coordinates of the stations were typed in, sent and within a split of seconds, the Geoid undulation values for the points were obtained. The geoid undulation values were subtracted from the ellipsoidal heights obtained from Global Navigational Satellite System-Precise Point Positioning (GNSS-PPP) observations to obtain the orthometric heights of the two GNSS-PPP points used. The orthometric heights then, served as the reduced level value for a point named "Kogi State Poly 005s" (KSP 005s) during the levelling operation including a fly back leveling. The redundant observations carried out were five (5) in numbers with a mean of 0.0461m and a standard deviation of 0.003m respectively. The results of the observations revealed that height accuracies. At 95% confidence level, the marginal error of the differences is $\pm0.0026m$. The results of the observations revealed that height accuracies provided by GNSS-PPP cannot be used in engineering applications where high accuracy is required. It is therefore recommended that local geoid be determined by gravimetric method to always check the applicability of EGM 2008.

Keywords: Ellipsoidal height, GNSS-PPP, Geoid undulation, Orthometric height

INTRODUCTION

Height as one of the results of Global Navigational Satellite System (GNSS) observation is the vertical distance from the surface of the earth referenced to the Geoid or ellipsoid (Hazelton 2020). GNSS-PPP heights are obtained via the use of a single GNSS receiver alongside orbit and clock data products which eliminate errors in the general positioning results (Guma *et al.,* 2023 and Lachapelle *et al.,* 2006). These errors include phase ambiguity terms, receiver clock offset, ionospheric delay and the tropospheric effect, etcetera (Petovello, 2015). GNSS-PPP are conveniently carried out via the internet and e-mail services after RINEX files of observations have been sent to the various PPP platforms (Kwiecien and Malinowski, 2016 and Guma, 2022). GNSS-PPP positions are referenced to a global reference frame, and for this reason, it delivers much better positioning consistency than the differential technique which has baseline limitations (Alkan, *et al.* 2016).

Unlike the differential GNSS positioning system (DGNSS) technique, Heights determined by GNSS-PPP technique are not relative to any localized base station which implies that no error is reflecting on all the new stations (Petovello, 2015). Guma *et al.,* (2023) agreed that, the GNSS-PPP technique reduces the large number of personnel and cost that are associated with the differential technique and by so doing; it simplifies the logistics in field operations. Unlike the height obtained through levelling which is always referenced to the geoid or mean sea level, the GNSS-PPP is always referenced to the ellipsoid (Hazelton 2020).

Among numerous uses of heights are their use in topographic analysis of the terrain (Meyer, Roman and Zilkoski, 2006) and geophysical analysis of the terrain are the most prevalent ones (Alexeyev *et al.* 1996). Heights could be useful in deformation surveys and also in guidance and monitoring of machines ((Deloney, 2022 and Kemboi and Odera, 2016). Mpabanyanka (2019) explained that heights could help Highway Engineers understand the sloppiness of the land which may allow for smooth movement of water and other liquids. Heights are vital in gravity measurements and its applicability in assessing the durability and functionality of buildings. By obtaining the heights of the ground, a contour map that displays the shape of the land could be produced and this terrain information could be used for different purposes such as environmental management, landuse and landcover planning and engineering projects (Burian *et al.,* 2004).

This study was prompted by the fact that, in the study area only GNSS-PPP controls are available and since it is an academic environment, projects could be assigned to students as part of their requirements for certification; so, the GNSS-PPP controls which are in Northing (N), Easting (E) and Height (H) could be used as starting controls and as well as closing controls (in levelling works) respectively. Therefore, the reliability and accuracy of these controls have to be ascertained. Several similar literatures were visited in preparation for this study but, none of them

talked about using of GNSS-PPP heights rather, they talked much about GPS (differential technique). The various Authors reviewed only wrote about orthometric heights derived from GNSS (differential GNSS or GPS technique) and levelling heights alone. Hence, this research aims at verifying the reliability of GNSS-PPP heights in engineering design with the objective of determining the range of misclosures or error limit offered by GNSS heights.

ASSESSING THE ACCURACIES OF GNSS HEIGHTS

Herbert and Olatunji (2020) carried out a comparative analysis of the differences in ellipsoidal heights and orthometric heights of points, with Differential Global Positioning System (DGPS) Satellite data for a duration of thirty (30) minutes each on 50 points of interest. The levelling was also performed on the same 50 points where the result shows that the accuracy of the height differences were 53.59cm for ellipsoidal height measurement and 53.07cm for orthometric height measurement. Herbert and Olatunji (2020) later inferred that, the two height systems should be used interchangeably for determination of differences in heights over short distances.

Badejo *et al.,* (2016), observed that the differences in heights in GNSS observation and spirit levelling are almost insignificant over a flat terrain as they satisfied third order levelling specifications especially when the results for Orthometric heights for the same set of points are determined using geodetic levelling. The height differences between GNSS and levelling heights had mean accuracy of 13.2ppm over a cumulative distance of 139.114km. The accuracy meets the requirement for engineering surveys, that is, $27mm\sqrt{k}$.

Herbert and Olatunji (2020) showed that there was no significant difference in the performance of the heights obtained with GNSS (ellipsoidal) and that obtained through the level (orthometric) height systems. It was advised that GPS and spirit levelling height differences be used interchangeably for short distances in surveying and engineering applications.

Malinowski and Kwiecien, (2016) did a comparative analysis of observations lasting between 1 – 7 hours which showed accuracy of 2 - 4cm. Malinowski and Kwiecien, (2016) inferred that, 1 – 2cm accuracy is feasible for both horizontal and differences in height measurements when duration for GNSS-PPP observations last for two hours.

Krzan *et al.,* (2015) analysed GNSS-PPP performance in the determination of normal heights via observations from an assessment network involving of 10 locations. After about 1 week of observation duration, the result were processed using the usual GNSS-PPP simulation that takes care of ambiguities alongside the PL–geoid–2011 model and the results of the research proved that, the GNSS-PPP heights are a possible options for Relative GNSS Positioning in the case of its comparison with GNSS levelling.

Abou-Galala *et al.,* (2018), wrote that the accuracy of GNSS-PPP was tried on several stations in 2012 and convergence times which are of different levels were studied too. It was however

inferred that, the static GNSS-PPP accuracy is among the range of 9mm in the vertical component (which is the height). GNSS-PPP according to Abou-Galala *et al.,* (2018) would require 1 hour to attain 5 cm horizontal accuracy.

Song and Zhao (2021) observed that, the hourly accuracy for the static GNSS-PPP was 5.6 mm, 9.2 mm and 12.6 mm in the north, east and the vertical direction. Galileo satellites showed the best convergence performance for single GNSS positioning, and the GPS/Galileo combined PPP achieved the best performance for the PPP using different GNSS combinations.

Lachapelle *et al.,* (2016) after investigating for two hours, some post-processing solutions such as Automatic Precise Positioning Service (APPS), Canadian Spatial Reference System Precise Point Positioning (CSRS-PPP), GNSS Analysis and Positioning Software (GAPS) and magicPPP - Precise Point Positioning Solution (magicGNSS), it was inferred that an accuracy of 2-4 cm were achieved.

Kemboi and Odera (2016) who adopted the WGS84 as the reference system, used the EGM 2008 geoid model as obtained via Alltrans EGM2008 calculator software version 3.002. In this study, the accuracy of ±0.52m and ±0.35 m were obtained respectively however, after the four parameter model correction was applied, accuracy was reduced to ±0.10 m, which represents about 71% improvement. The accuracy of ±0.10 m obtained at the test points may be sufficient for some engineering projects that do not require very high orthometric height accuracy.

GNSS-PPP Principle

Lachapelle *et al.,* (2006) presented that, position determination with GNSS-PPP has been based on ionospheric free combinations of the undifferenced code and phase observations called the traditional mode;

$$P_{IF} = \frac{P_I^2 \cdot P(L1) - f_2^2 . P(L2)}{f_1^2 - f_2^2} = \rho - cdT + d_{trop} \qquad (1)$$

$$P_{IF} = \frac{P_I^2 \cdot \emptyset(L1) - f_2^2 . \emptyset(L2)}{f_1^2 - f_2^2} = \rho - cdT + d_{trop} + \frac{cf_1 N_1 - cf_2 N_2}{f_1^2 - f_2^2} \qquad (2)$$

Where $f_1 \ and \ f_2$ are the GPS L1 and L2 frequencies; $P(L1)$, $\emptyset(L1)$ are the code and phase observations; ρ is the true geometric range; c is the speed of light; dT is the receiver clock offset; dtrop is the tropospheric effect; N_i is the phase ambiguity term in $\emptyset(L1)$.

According to Equations (1) and (2) the unknown parameters to be estimated in GNSS-PPP include position coordinates, phase ambiguity terms, receiver clock offset and the tropospheric

effect. These precise orbit and clock products that could produce centimetre-level accuracy in GNSS-PPP techniques are accessible both in post-mission and real-time. Other ambiguities such as Phase wind-up correction, satellite antenna offset, and site-displacement effects due to solid earth tide and ocean loading are part of what the GNSS-PPP removes to achieve the desired results.

Ayodele *et al.,* (2018) simplified the equation 1 and 2 with four (4) distance measurements from the satellite which all ended at the same point which is, the location of the receiver. All the satellite positions, clock biases, and more, are contained in the satellite broadcast ephemerides and all these data are incorporated in the actual measurement streams; position estimate is limited in accuracy if measurement corrections are not applied (Carlin *et al.,* 2022). With the four (4) distance measurements and the satellite positions, the receiver can determine its location and its clock offset. The model for such is denoted in equations 3a to 3d.

$$d_1 = \sqrt{(x - x_1)^2 + (y - y_1)^2 + (z - z_1)^2} + ct_B \qquad \text{3a}$$

$$d_2 = \sqrt{(x - x_2)^2 + (y - y_2)^2 + (z - z_2)^2} + ct_B \qquad \text{3b}$$

$$d_3 = \sqrt{(x - x_3)^2 + (y - y_3)^2 + (z - z_3)^2} + ct_B \qquad \text{3c}$$

$$d_4 = \sqrt{(x - x_4)^2 + (y - y_4)^2 + (z - z_4)^2} + ct_B \qquad \text{3d}$$

Levelling

Levelling is the process of determining the difference in height between two points by the use of a level. There are many other ways of determining heights or elevation of points but, when it is done with a level instrument, it becomes levelling (Liu, 2022). In this study, levelling was deployed in transferring heights from one station to the other. Though there are different types of levelling operations, the differential levelling was used to achieve the aim of this study. There are ways of evaluating the permissible errors in levelling work and also, this accuracy is dependent on the carefulness of the Survey personnel involved (Uren and Price, 2019). The formula is denoted as;

$$E = C\sqrt{D} \qquad 4$$

For various categories of levelling work, (Anupoju, 2016) expressed their permissible misclosures or errors in equations 5 to 8 respectively.

$$\text{Rough levelling, } E = \pm 0.100\sqrt{D} \qquad 5$$

$$\text{Ordinary levelling, } E = \pm 0.025\sqrt{D} \qquad 6$$

$$\text{Accurate levelling, } E = \pm 0.012\sqrt{D} \qquad 7$$

Precise levelling, E=±0.006√D   8

Where, E = misclosure in meters, C is the constant, and D is the distance in kilometres. Misclosures permissible for engineering works are usually in the class of ordinary leveling. In Nigeria, it has been adopted to be;

$$E = \pm 27mm\sqrt{k} \qquad\qquad 9$$

where k, is the distance covered in kilometres which is same as D in other equations. The 27mm is 0.0027m which almost equalled the ordinary level permissible error range (SURCON, 2007). The booking and adjustment of levelling operation is done by either Height of collimation method or rise and fall method. For this study, the rise and fall method is used.

Geoid Model

Geoid models are a creation of the National Geospatial intelligence Agency (NGA) and are meant to provide mean sea level everywhere on the surface of the earth (Peprah *et al.,* 2017). Accurate geoid model is necessary for the determination of orthometric heights especially when using the GNSS technique at the local scale (ibid). Geoid is very vital in the determination of orthometric height from ellipsoidal heights especially when the expert is using GNSS technique. The separation between the ellipsoid and the geoid gives the geoid undulation, see figure 1.
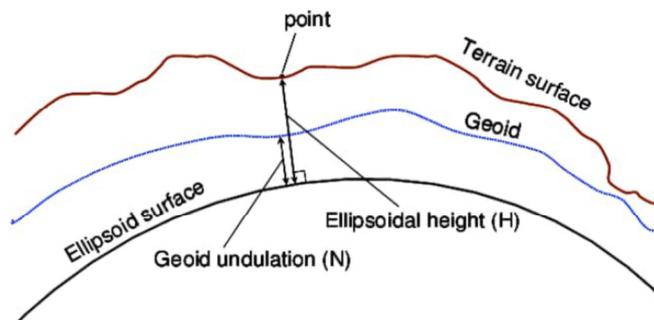

Figure 1: The relation between ellipsoidal height and EGM 2008 (Geoid undulation)

In geodesy, 3 surfaces exist and they are the earth, geoid, *h* and the ellipsoidal surface, *H.* The relationship among them is such that;

$$H = h - N \qquad\qquad 10$$

The *h,* is the ellipsoidal height, the *H,* is the orthometric height and the *N,* is the geoidal undulation. The value N, is what is being supplied by EGM 2008 and could be obtain by gravimetric means, astrogeodetic means and, or satellite geometric method (Kemboi & Odera, 2016). Fadi and Sideris (2003) also described two accurate methods that geoid could be obtained from and they are stoke's and Hotline's integral methods.

The stoke and Hotline integral methods are very tedious and rigorous to use in geoid determination that is why, the EGM 2008 geoid model is a better option today (Kemboi and Odera, 2016). The EGM 2008 model could be used to calculate the height difference between the ellipsoid and the Mean Sea Level on any place on the globe. The EGM 2008 geoid undulation formula as used by (Kemboi and Odera, 2016) is presented in equation 11.

$$N_{EGM\ 2008} = \frac{GM}{r\gamma} \sum_{n=2}^{n_{max}} \left(\frac{a_{ref}}{r}\right) \sum_{m=0}^{n} (\bar{C}*_{nm}\ cosm\lambda + \bar{S}_{nm} \sin m\lambda) \bar{P}_{nm}(cos\theta) \qquad 11.$$

Where $N_{EGM\ 2008}$ is the geoid-ellipsoid separation $\bar{P}_{nm}(cos\theta)$, is the fully normalized Legendre function of degree n and order m, $\bar{C}*_{nm}\ and\ \bar{S}_{nm}$ are spherical harmonic coefficient, a is equatorial radius of adopted reference ellipsoid, GM is the gravitational constant, a_{ref} is the scaling parameter, r is the mass of the earth, n_{max} is the finite maximum degree of a GGM.

To further determine the geoid undulation of points, the three surface equation is used. To obtain the orthometric height of any point the equation 11 is used;

$$H_{MSL} = h_{WGS\ 84} - N_{EGM\ 2008} \qquad\qquad 12$$

Where H_{MSL}, the Height above mean sea level or geoid is, $h_{WGS\ 84}$ is the height above ellipsoid and $N_{EGM\ 2008}$ is the geoid-ellipsoid separation for 2008 EGM.

METHODOLOGY

Study Area

The project area is within Kogi State Polytechnic Lokoja campus in Lokoja L.G.A, Kogi State and lies between latitudes 7° 45' 27.56" N to 7° 51' 04.34" N and longitudes 6°41' 55.64" E to 6°45' 36.58" E; The town is situated in the tropical Wet and Dry savannah climate zone of Nigeria, and temperature remains hot all year round (Guma *et al.*, 2023). The reason for the study area was because there was a need to verify the accuracy expected any time practical works are carried out by the students. The GNSS-PPP controls were obtained by GNSS-PPP technique and the observation duration was 2 hours.
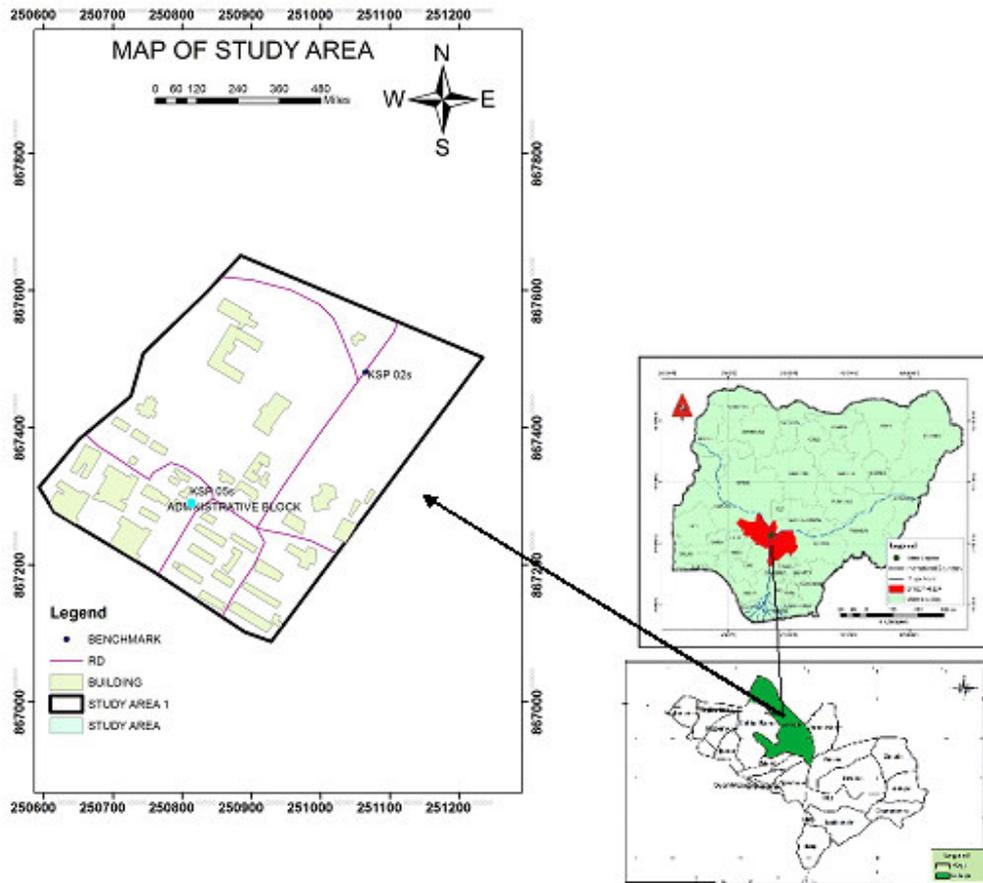
Figure 2: Map of the Study Area Showing Location within Kogi State and Nigeria
Source: Field work, 2023

Instrumentation and data needed

The instruments used for this study were Automatic level and its accessories together with GNSS V30 Hi-Target receiver with its accessories. The primary data used for this research were the levelling operations results. The secondary data used were two GNSS-PPP coordinates which included the ellipsoidal heights and the geoidal undulation values for the two GNSS-PPP stations.

Method of Primary data acquisition

Levelling data

In this study, before the levelling data were acquired, a 2-peg-test was performed to ascertain the state of the level and the result was in 2 mm. During this field operation, backsights and foresights were observed on levelling staff held on points of interest usually 20m apart to check

for errors which might likely occur as a result of refraction, curvature and collimation. The first sight was taken by observing the staff through the telescope of the level to read. So first observation reading booked while the staff was held on a known height is what is known as the Backsight reading. Other observations before the last one that would make instrument change of position is called intermediate reading. In this study, the intermediate sights were never observed. Meanwhile, the last observation carried out before change of instrument position is called foresight. Backsights and foresights were observed in the field during primary data acquisition. At the beginning of the observation, the levelling staff was held on station "KSP 005s" which served as the Bench Mark (BM). The Foresight reading was observed at the levelling Staff held on station "KSP 002s". These two points are 382.969m apart. For check purpose, a fly back levelling was performed back from "KSP 002s" to station "KSP 005s". The reason for using these two control points ("KSP 002s" and "KSP 005s") is because they are GNSS-PPP stations observed on static mode for 2 hours each on different days. The method of differential levelling was used.
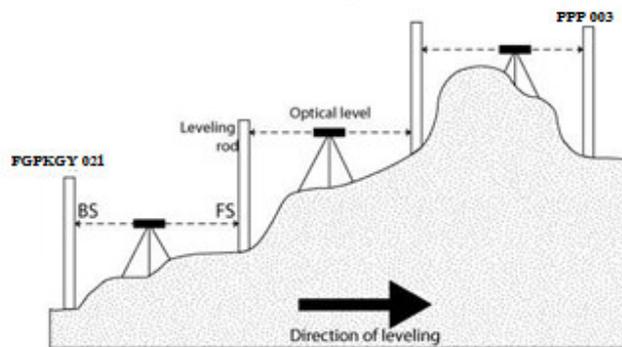

Figure 3: Levelling field work.

Method of Secondary data acquisition (GNSS-PPP heights Observations)

The Hi-target V30 GNSS receiver was used to acquire satellite signals at 2 hour interval for each of the two stations acquired. However before its usage, a temporary adjustment was conducted and the plate bubble test was conducted to be sure of the state of the receiver. The GNSS receiver was mounted on the each of the two control stations and after a 2 hour each duration of satellite signal acquisitions, the data were processed using Hi-Target Geomatics office (HGO) software the Canadian Spatial Reference System (CSRS). The post-processed results were obtained via the internet.

Method of acquisition of EGM 2008 values

The geoidal undulation values of EGM 2008 were obtained using online GeoidEval utility software sourced from Karney (2022). The geoid eval utility software computes the geoidal height using interpolation in grid of values. The RMS usually obtained after the interpolation is in

the range of 1mm (Karney, 2022). At first, the geographical coordinates of the two control stations (KSP 005s and KSP 002s) were typed in a form that was displayed. A submit button was clicked and within a split of seconds, the Geoid undulation values for the two stations were interpolated and displayed.

DATA ANALYSIS

Levelling data

The Levelling data are hereby tabulated from Tables 2-6 respectively, and the Tables contain the back-sight and Foresights reading only. There is the column for rise and fall because that was the method that was used for the processing of data. The height value used to compute the levelling data is the orthometric height obtained from the EGM 2008 GeoidEval values in Table 1. The calculations under the Tables 2 to 6 are called checks for Levelling observations: After every levelling operations, checks are usually carried out to ascertain the accuracy of such operations; where, summation of Backsight (BS) stadia reading minus summation the of Foresight stadia (FS) reading MUST be equal to summation Rise Minus that of Fall and must also be equal to Last Reduced Level (LRL) minus First Reduced Level (FRL).

Table 1: Values of ellipsoidal height, geoid undulation and orthometric heights

STN ID	Ellipsoidal height H (m)	Geoid undulation (EGM 2008) N (m)	Orthometric height H (m)
KSP 005s	95.166	23.3443	71.8217
KSP 002s	83.443	23.3444	60.0986
KSP 003s	95.134	23.3437	71.7903

Source: Fieldwork, 2023

Table 2: First levelling Observation

Backsight (m)	Foresight (m)	RISE(m)	FALL (m)	Reduced Level (m)	REMARK	Given Coordinate (m)	Diff. (m)
0.850				71.8217	KSP 005s		
1.800	1.715		0.865	70.9567			
0.916	0.880	0.92		71.8767	KSP 003s		
0.709	2.905		1.989	69.8877			
0.550	2.322		1.613	68.2747			
0.552	2.500		1.950	66.3247			
0.566	2.338		1.786	64.5387			
0.290	2.255		1.689	62.8497			
1.274	2.848		2.558	60.2917			
	1.424		0.150	60.1417	KSP002s	60.0986	0.0431
$\sum BS$ =7.507	$\sum FS$ =19.187						

Source: Fieldwork, 2023

$$\sum BS - FS = 7.507 - 19.187 = -11.680 \ m$$
$$Diff. btw \ KSP \ 002s - KSP \ 005s = 60.1417 - 71.8217 = -11.680$$

Table 3: Second levelling observation

Backsight (m)	Foresight (m)	RISE(m)	FALL (m)	Reduced Level (m)	REMARK	Given Coordinate	Diff. (m)
0.868				71.8217	KSP 005s		
1.86	1.745		0.877	70.9447			
0.971	0.93	0.93		71.8747	KSP 003s		
0.63	2.96		1.989	69.8857			
0.69	2.24		1.61	68.2757			
0.671	2.645		1.955	66.3207			
0.58	2.456		1.785	64.5357			
0.491	2.265		1.685	62.8507			
1.224	3.042		2.551	60.2997			
	1.374		0.15	60.1497	KSP 002s		**0.0511**
∑ **BS** =7.985	∑ **FS** =19.657						

Source: Fieldwork, 2023

$$\sum BS - FS = 7.985 - 19.657 = -11.672 \ m$$
$$Diff. btw \ KSP \ 002s - KSP \ 005s = 60.1497 - 71.8217 = -11.672 \ m$$

Table 4: Third levelling observation

Backsight (m)	Foresight (m)	RISE(m)	FALL (m)	Reduced Level (m)	REMARK	Given coordinate	Diff. (m)
0.915				71.8217	KSP 005s		
1.864	1.79		0.875	70.9467			
0.965	0.934	0.93		71.8767	KSP 003s		
0.819	2.959		1.994	69.8827			
0.612	2.43		1.611	68.2717			
0.171	2.566		1.954	66.3177			
0.189	2.526		2.355	63.9627			
0.493	2.55		2.361	61.6017			
	1.952		1.459	60.1427	KSP 002s		**0.0441**
∑ **BS** =6.344	∑ **FS** =18.019						

Source: Fieldwork, 2023

$$\sum BS - FS = 6.028 - 17.707 = -11.679 \ m$$
$$Diff. btw \ KSP \ 002s - KSP \ 005s = 60.1427 - 71.8217 = -11.679 \ m$$

Table 5: Fourth levelling observation

Backsight (m)	Foresight (m)	RISE(m)	FALL (m)	Reduced Level (m)	REMARK	Given coordinate	Diff. (m)
0.938				71.8217	KSP 005s		
1.869	1.78		0.842	70.9797			
0.635	0.969	0.9		71.8797	KSP 003s		
0.611	2.624		1.989	69.8907			
0.693	2.605		1.994	67.8967			
0.374	2.61		1.917	65.9797			
0.74	2.871		2.497	63.4827			
0.484	2.422		1.682	61.8007			
	2.138		1.654	60.1467	KSP 002s		**0.0481**
∑ **BS** =6.344	∑ **FS** =18.019						

Source: Fieldwork, 2023

$$\sum BS - FS = 6.344 - 18.019 = -11.675 \ m$$
$$Diff.\,btw\ KSP\ 002s - KSP\ 005s = 60.1467 - 71.8217 = -11.675 \ m$$

Table 6: Fifth and Fly-back levelling observation for check

Backsight (m)	Foresight (m)	RISE(m)	FALL (m)	Reduced Level (m)	REMARK	Given Height (m)	Diff. (m)
2.138				60.0986	KSP 002s		
2.011	0.374	1.764		61.8626			
2.108	0.909	1.102		62.9646			
2.502	0.935	1.173		64.1376			
2.241	0.745	1.757		65.8946			
2.248	0.95	1.291		67.1856			
2.109	0.921	1.327		68.5126			
2.327	0.761	1.348		69.8606			
0.809	0.361	1.966		71.8266	KSP 003s		
1.691	1.734		0.925	70.9016			
	0.815	0.876		71.7776	KSP 005s		**0.0441**
∑ **BS** =20.184	∑ **FS** =8.505						

Source: Fieldwork, 2023

$$\sum BS - FS = 20.184 - 8.505 = 11.679 \ m$$
$$Diff.\,btw\ KSP\ 005s - KSP\ 002s = 71.7776 - 60.0986 = 11.679 \ m$$

Table 7: Summary of the difference between levelling (observed) height and EGM 2008 derived Orthometric heights

Height ID	First levelling	Second levelling	Third levelling	Fourth levelling	Fly-back/ fifth levelling
Orthometric height obtained from levelling	60.1417	60.1497	60.1427	60.1467	71.7776
EGM 2008 derived orthometric height	60.0986	60.0986	60.0986	60.0986	71.8217
Differences	0.0431	0.0511	0.0441	0.0481	-0.0441

Source: Fieldwork, 2023

Levelling Result Analysis

For the purpose of finding the accuracy of this work, the last column, will be taken as positive. We are only interested in the difference without the negative sign with it. These differences were subjected to statistical analysis.

Mean and standard deviation of the misclosures

The mean of the differences (misclosures) is presented as;

$$Mean = \frac{\Sigma x}{n} \qquad\qquad 13$$

=0.0461 m

Standard deviation of the differences (misclosures)

$$\sigma = \sqrt{\frac{\Sigma(X-\mu)^2}{N}} \qquad\qquad 14$$

Where σ is the standard deviation, X signify each value and μ is the mean and N, is the total number of times the observations were carried out and of course, the square root of all of them. $\therefore$ Standard Deviation, $\sigma = 0.0030331501776206 = 0.003m$

Margin of Error (Confidence Interval)

The standard error of the mean (SEM) is calculated as thus;

$$\sigma_x = \frac{\sigma}{\sqrt{N}} = 0.00135646 \qquad\qquad 15$$

Based on the SEM, the following are confidence intervals at different confidence levels; however, a confidence level of 95% (or statistical significance of 5%) is typically used for data representation in surveying and geodesy discipline. Table 8 displays all the various confidence levels and their margin of errors.

Table 8: Summary of the confidence level

Confidence level	Margin of error
68.3%, $\sigma_{\bar{x}}$	0.0461 ±0.00136 (±2.94%)
90%, 1.645$\sigma_{\bar{x}}$	0.0461 ±0.00223 (±4.84%)
95%, 1.960$\sigma_{\bar{x}}$	0.0461 ±0.00266 (±5.77%)
99%, 2.576$\sigma_{\bar{x}}$	0.0461 ±0.00349 (±7.58%)
99.9%, 3.291$\sigma_{\bar{x}}$	0.0461 ±0.00446 (±9.68%)
99.99%, 3.891$\sigma_{\bar{x}}$	0.0461 ±0.00528 (±11.45%)
99.999%, 4.417$\sigma_{\bar{x}}$	0.0461 ±0.00599 (±13.00%)
99.9999%, 4.892$\sigma_{\bar{x}}$	0.0461 ±0.00664 (±14.39%)

Source: Fieldwork, 2023

DISCUSSION

The Tables 2 to 6 were levelling observation bookings of the same points. In order to establish any claim, data consistency needs to be established. Redundant observations were carried out and the first one had a misclosure of 0.0431m even after the necessary checks were performed. The various checks were performed proved that the levelling work was in order. In the first levelling operation, instead of the levelling to close at 60.0986m, it closed at 60.1417m with a difference of 0.0431m. The second levelling operation closed at 60.1497m instead of 60.0986, leaving a difference of 0.0511m. Again, the third one closed at 60.1427m with a difference of 0.0441m. Finally, a fifth levelling operation was carried out which was in the form of a flyback levelling which started from the final point and closed at the usual beginning point, KSP 005s. The flyback levelling closed at 71.7776m instead of 71.8217m thereby producing a difference of 0.0441m. See table 7 for the summary of the various differences.

The mean of the differences as calculated is 0.0461m which is relatively close to the individual differences obtained. The standard deviation and the 95% confidence level of the differences are 0.003m and 0.0461±0.00266m respectively. These values statistically show that the observations were highly precise. However, there was concern on control KSP 003s which is 74.778m away from KSP 005s. These two points have a difference in the ellipsoidal heights as 0.032m, and a difference in their orthometric heights as 0.0314m. However, when these points were included in the levelling table, their difference was averagely 0.05m. The surprising fact is that, it was expected that by the closeness of these points to one another, all their orthometric heights differences, ellipsoidal height differences and the levelling table orthometric heights ought to be of the same value. But, it could be inferred that, the reason for this height differences between KSP 005s and KSP 003s is because there could be rapid change in the geoid-ellipsoid separation which is the geoid undulation passing underneath that area.

Finally, the mean of the 5 differences could be used to formulate the permissible misclosure for any leveling operation that may want to use these GNSS-PPP controls.

If we have; $0.0461 = \pm y\sqrt{0.382969}$

Then, $y = \frac{\sqrt{0.382969}}{0.0461}$

Therefore, the misclosure for any levelling work within the campus can be checked for permissible error with the formula;

$$E = \pm 0.074 \sqrt{k} \qquad\qquad 15$$

Where E is the permissible misclosure (error) and k is the distance in kilometres and 0.074 is the derived constant in GNSS-PPP height of 2 hours observations. This formula will be used for any GNSS-PPP controls of 2 hours static observation.

CONCLUSION

It has been shown that the GNSS-PPP derived orthometric heights cannot be relied on in engineering design or construction; the reason being that, in some areas or routes, it has been affirmed that, there used to be sharp change in the geoid-ellipsoid separation. In some places, this rapid change may not occur, but there is no certainty as at where to expect this sharp change that is able to affect accuracies. It can be concluded that the accuracy obtained after the statistical analysis, is better than that obtained by Kwiecien (2016), who had 0.2 m as accuracy, Lachapelle *et al.*, (2016) who had 0.2 m to 0.4 m accuracy, and Kemboi and Odera (2016) who had their accuracy between 0.52 m and 0.35 m before corrections were applied. The effect of the rapid change in the geoid-ellipsoid separation is responsible for the misclosures obtained in the 5 different levelling operations embarked upon in this study therefore, this work agrees with Kearsley (1988) and Kemboi and Odera (2016) who discouraged the use of this height in engineering works. More so, equation 15 was determined as a contribution to knowledge and to be used to check accuracies of levelling works whose Benchmark values are obtained via 2 hours GNSS-PPP static observations. Moreover, this study is work in progress since the equation will be used to check the misclosures of student levelling operations that will soon be carried out in the campus. It is therefore recommended that GNSS-PPP be only use for third order survey, but definitely not for pipelines surveys nor road construction survey where accuracy needed is high. Again, it is recommended that gravimetric geoid be determined to always use to check the applicability of EGM 2008 geoid.

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