Densification of First-Order Geodetic Controls within the University of Benin, Nigeria

Ehigiator-Irughe R. and Oladosu S. O.*

Department of Geomatics, Faculty of Environmental Sciences,
University of Benin, P.M.B. 1154, Edo State, Nigeria.

Email: raphael.ehigiator@uniben.edu
*Email: olushola.oladosu@uniben.edu

Abstract

This study focused on the densification of first-order controls within the University of Benin Ugbowo campus, using CORS_Geosystems as the virtual reference station. The evolving developmental project within the campus necessitated the establishment of more control points. Eight newly designated control points were designated for coordination. In addition, one previously established control point with the ID Raph_UB_02 was re-coordinated to validate the accuracy of the positional data collected compared to the pre-existing record. The rover receivers' occupation time at each control point lasted at least one hour. The downloaded data was processed and adjusted using Trimble Business Centre 2.70. The maximum 2D and 3D precisions obtained were 1:6,038,657 and 1:6,035,931 along the CORS_Geo and Raph GNSS_05 baselines, while the minimum 2D and 3D precisions obtained were 1:4,255,886 and 1:4,256,390 along the CORS_Geo and RAPH GNSS_11 baselines. All the calculated values for the allowable limit in the Class "A" survey for control densification had a 3D maximum standard error of 0.011 m. These results are more accurate than the recommended standard by the Surveyors Council of Nigeria. Thus, this study provided a piece of verifiable evidence about the CORS_Geosystems' efficacy in providing quality positioning solutions at higher accuracy and the suitability of the established control points on campus for various applications. These controls are therefore recommended for precise engineering projects and other developmental applications.

Keywords: *Adjustment, Accuracy, CORS, Control densification, GNSS*

Introduction

CORS is a short acronym for "Continuously Operating Reference Station." CORS can be operated as a single station or multiple network stations, and it can run nonstop for 24 hours on a constant power supply (Leica Geosystems, 2005). The installation of CORS is such that it can continuously function as a reference station for GNSS observation. Its components include a computer system, data communications devices, and internet (LAN/WAN) technologies.

Authorised users are automatically given access to various GNSS observation data solutions in the pseudo-range or carrier phase for (both) static and real-time modes. It also provides various corrections, measurements, and other GNSS service systems in real-time with centimetre-level or millimetre-level precision achieved in the National Spatial Reference System in many countries (Wu et al., 2015; Leica Geosystems, 2005).

It is now much easier for geomatics engineers and surveyors to densify controls at necessary locations for use in engineering project execution, smart city and urban planning, environmental disaster monitoring, hazard mitigation, structural deformation, and subsidence monitoring, utilities as-built survey, project management, oil and gas geomatics support, and other applications.

In recent years, the University of Benin has witnessed some level of developmental stride in structural and infrastructural within the Ugbowo campus. To properly monitor and execute multi-million projects successfully, adequate control points are very important. According to Wu et al. (2015), the practice of densifying first-order control with the help of DGNSS or CORS is widespread. It has been approved as a practising standard by the Surveyors Council of Nigeria (SURCON, 2007).

For the first-order and second-order controls across Nigeria, researchers have previously used GNSS and CORS (Iyiola et al., 2013; Ehigiator et al., 2017; Ojigi, 2015; Ehigiator & Oladosu, 2017; Udochukwu et al., 2019). SURCON recommend that only a zero-order (satellite-based system) or another first-order control equivalent with a track record of integrity should be used to establish these first-order controls. First-order GNSS control networks are required at crucial locations such as the university community to enable precise position determination and monitoring of structures within the campus in an effective, efficient, and aesthetic manner.

According to Jatau et al. (2010), the history of CORS installation in Nigeria is traceable to the year 2008, when the Office of the Surveyor General of the Federation (OSGoF) installed some CORS in selected areas across the country. To provide access to high-accuracy data from GNSS virtual controls, the GNSS geodetic network and its reference frame must be checked regularly and periodically (Ezeigbo, 2004).

In line with Ezeigbo (2004) recommendation, a GNSS routine check campaign was conducted from October 2010 to April 2011, as reported by (Jatau et al. 2010; OSGoF, 2010) with the main objective of checking the existing network and the possibility of strengthening and expanding the coverage of the network to accommodate new stations up to sixty monitored for 48 hours. These stations were spread throughout the GNSS network to link the Nigerian Primary Triangulation Network to the Zero Order Geodetic Network (NIGNET), thereby leading to the evolution of a new Nigerian Primary Geodetic Network (NPGN) based on the NGD 2012 reference frame. Ayodele et al. (2020) assessed the NIGNET archival data from 2011 to 2016 and found that the standard errors (SE) and root mean square errors (RMSE) ranged from 13.00 - 56.50 mm and 14.38 - 73.16 mm respectively, in line with the IGS standards, the results signify a high accuracy of about 88% with the rest 12% error attributed to missing data.

In surveying measurements, either ground-based or satellite-based, there are inherent errors. Therefore, the issue of error is also fundamental in GNSS observations, and the magnitude contributed by each media must be accounted for and statistical analysis enforced to make the final position solution acceptable (Shirazian et al., 2020; Ghilani & Wolf, 2012). Karaim et al. (2018), pointed out some of the errors that affect GNSS observation and the ways to minimise them widely covered.

This study highlighted one of the many advantages derived from the previously installed CORS_Geosystems in Benin City (Oladosu et al., 2022). It is a privately owned station serving as a reliable tool for control densification and mapping in the available modes of operation. Therefore, extending control points within the University of Benin Ugbowo campus vis a vis the results obtained will convince whether the single CORS network system is fulfilling its purpose of installation. These newly established controls will serve as an infrastructure for the subsequent checking, coordination, development, execution, and monitoring of existing and prospective engineering projects, and they will continue to be explored as reference points for future extension of more controls and the acquisition of geospatial data for mitigating against other environmental challenges.

Methodology

Study Area

The study area is located within Benin City, the Edo State capital. It is in Zone 31, North of the UTM projection. The coordinates are: 785989.43 mE; 701391.49 mN and 792152.39 mE; 700805.28 mN. Figure 1 is the map of Nigeria showing Edo State and the University of Benin, the study area. The CORS_Geosystems can cover the three Local Government Areas (LGAs) making up the Benin City metropolis and beyond, having a circumference encompassing about 70 km for RTK positioning mode. A distance (coverage) of up to 200 km or more on Static positioning mode can still afford users the signal reception for geospatial data acquisition.

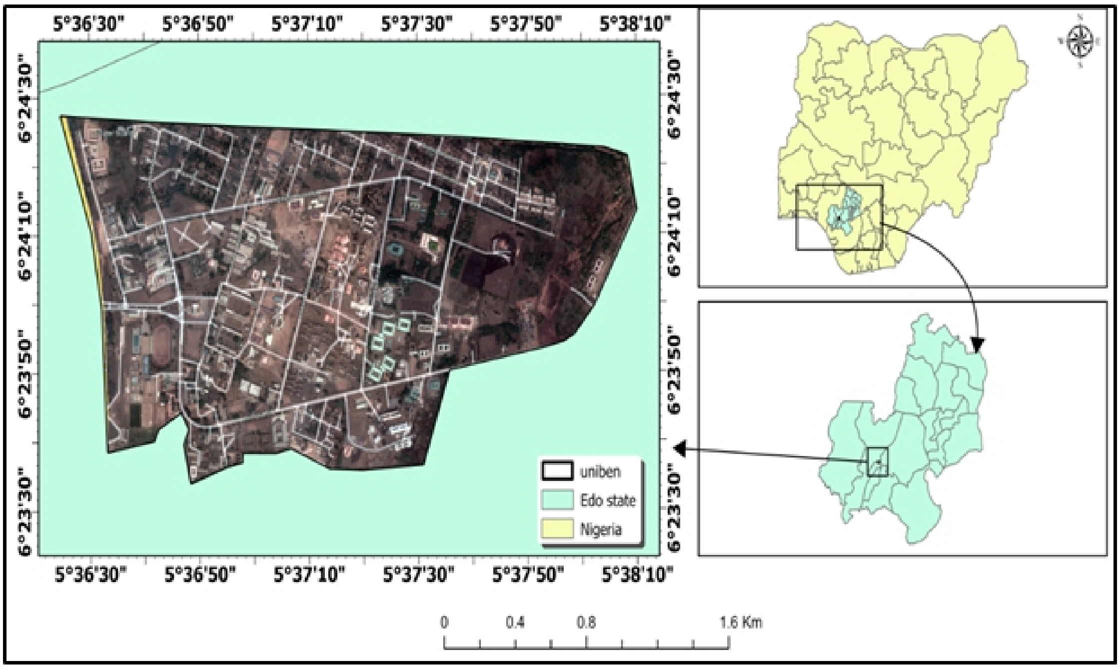
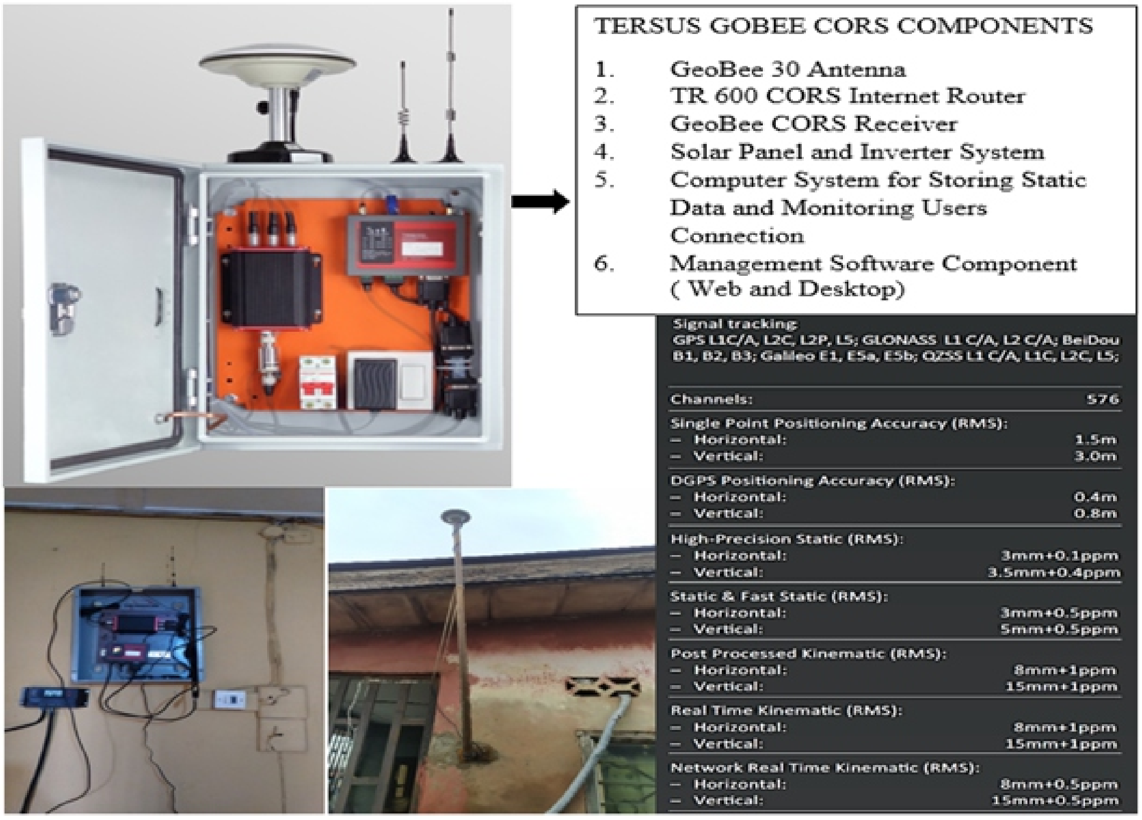


Figure 1: Map of the Study Area

Delivery Capacity of the CORS_Geosystems

As part of the effectiveness of the CORS_ Geosystem, which is a product of the Sacredion Tersus GeoBee 30 CORS is capable of providing seven position solutions in different modes with their stated accuracies, respectively. Figure 2 shows the components of the, its positional accuracies in different modes, and the location of its antenna point and connection systems. The effectiveness of the installed CORS_Geosystems in quality and accurate spatial data collection has been proven and presented in earlier research by (Oladosu et al. 2022).



TERSUS GOBEE CORS COMPONENTS

1. GeoBee 30 Antenna
2. TR 600 CORS Internet Router
3. GeoBee CORS Receiver
4. Solar Panel and Inverter System
5. Computer System for Storing Static Data and Monitoring Users Connection
6. Management Software Component (Web and Desktop)

Signal tracking
GPS L1C/A, L2C, L2P, L5; GLONASS L1 C/A, L2 C/A; BeiDou B1, B2, B3; Galileo E1, E5a, E5b; QZSS L1 C/A, L1C, L2C, L5;

Channels:	576
Single Point Positioning Accuracy (RMS):	
– Horizontal:	1.5m
– Vertical:	3.0m
DGPS Positioning Accuracy (RMS):	
– Horizontal:	0.4m
– Vertical:	0.8m
High-Precision Static (RMS):	
– Horizontal:	3mm+0.1ppm
– Vertical:	3.5mm+0.4ppm
Static & Fast Static (RMS):	
– Horizontal:	3mm+0.5ppm
– Vertical:	5mm+0.5ppm
Post Processed Kinematic (RMS):	
– Horizontal:	8mm+1ppm
– Vertical:	15mm+1ppm
Real Time Kinematic (RMS):	
– Horizontal:	8mm+1ppm
– Vertical:	15mm+1ppm
Network Real Time Kinematic (RMS):	
– Horizontal:	8mm+0.5ppm
– Vertical:	15mm+0.5ppm

Figure 2: Installation of CORS_Geosystems, Specifications, and Positional Accuracies (Modified from Tersus GeoBee30 User Manual, 2021)

Datum Transformation

In Nigeria, locations are calculated based on (both) geographic and rectangular coordinate systemS. With respect to the ellipsoid employed for geodetic computation in Nigeria, (Clarke 1880), the rectangular coordinates of points are computed in either the Nigerian (Modified) Transverse Mercator (NTM), Universal Traverse Mercator (UTM), or (by) both. Each grid system has its unique set of characteristics, and these attributes are used to calculate the position (Ehigiator-Irughe & Audu, 2016).

The Nigeria Minna datum, according to Ehigiator et al. (2011),is a geodetic datum suited for onshore and offshore applications in Nigeria. The Clarke 1880 (RGS) ellipsoid of the Minna Datum used in Nigeria has the following parameters: Semi major axis, a = 6378249.145m; Flattening, f = 1/293.465; Longitude: 6°30'58.76"E; Latitude: 9°38'08.87"N; orthometric height, H = 281.13 meters (Uzodinma et al., 2013). The processing of DGPS observations on the Minna datum involves the following:

Conversion of geodetic coordinates (latitude φ, longitude λ, and ellipsoidal height *h,*) on the WGS84 datum/ellipsoid to (latitude φ, longitude λ, and ellipsoidal height *h,*) on Nigeria Minna datum.

Conversion of geodetic coordinates (latitude φ, longitude λ, and ellipsoidal height h,) on the Minna datum/ellipsoid to Cartesian rectangular coordinates on the local datum, Minna datum.

Conversion of the geodetic coordinates (φ, λ, h) to plane rectangular systems, Nigeria Traverse Mercator (NTM) and Universal Traverse Mercator (UTM) coordinates.

Equations (1) to (6) can be applied for the purpose of coordinates transformation.

$$\left.\begin{aligned} X &= (N+h)\cos\varphi\cos\lambda \\ Y &= (N+h)\cos\varphi\sin\lambda \\ Z &= [N(1- \ e^2+h]\sin\varphi \end{aligned}\right\} \qquad (1)$$

Where: (φ, λ, h) are (respectively) the geodetic latitude, geodetic longitude and ellipsoidal height, while X, Y, and Z are the Cartesian coordinates to be estimated. h is the ellipsoidal height (orthometric height, H + geoidal height, N). N in Equation (1) is the radius of curvature in the prime vertical given by (Ono, 2009; Eteje et al. 2019) as:

$$N = \frac{}{(1- \left(2f- \ f^2 sin^2 \varphi\right)^{\frac{1}{2}}} \qquad (2)$$

Where a is the semi-major axis, b is the semi-minor axis, and f is flattening given as:

$$f = \frac{a-b}{a} \qquad (3)$$

The conversion of the geodetic coordinates on the global ellipsoid to Cartesian positions still on the global datum is necessary to transform the coordinates to positions on a local datum/ellipsoid using the seven datum transformation parameters.

The constants a and f are the dimensional parameters of (either) the regional or geocentric ellipsoids. In local ellipsoids, the parameter h is unknown. However, suppose Geoid-ellipsoid separation (Geoidal Undulation) and orthometric height (H) are known. In that case, the relationship between orthometric and geoidal height can be used to find h, as given by (Heiskanem and Morizt, 1967).

$$h = H + N \qquad (4)$$

Where: H = orthometric height, N = Geoid-ellipsoid separation (not to be confused with THE prime vertical radius of curvature N), h = ellipsoidal height

Conversely, Cartesian coordinates can be converted to geodetic coordinates, which may involve an iterative procedure to realise latitude (φ). Thus, from Equations (1), (2), and (3), a close solution is obtained as contained in Equation (5) (Featherstone and Vanicek, 1999):

$$\lambda = tan^{-1}\frac{Y}{X}$$
$$\varphi = tan^{-1}\left[\frac{1}{1-f}\tan u\right] \qquad (5)$$

$$\text{Where u} = tan^{-1}\left(\frac{1}{1-f}\frac{Z}{\sqrt{X^2+Y^2}}\right)$$
$$and\ h = \frac{X^2+Y^2}{\cos\varphi} - N$$

The Transformation Between WGS84 and Minna Datum

Processing the DGPS observations acquired on the WGS84 ellipsoid to obtain positions on the Minna datum/Clarke1880 ellipsoid requires datum transformation. This is because GNSS uses the WGS84 ellipsoid while the target datum is a local one with a different ellipsoid that best fits the region of application, for instance, the Minna datum. The accurate transformation of positions on the WGS84 ellipsoid to Minna datum, Clarke 1880 ellipsoid, requires the application of the seven datum transformation parameters. The application of the seven datum transformation parameters requires their combination with the Cartesian coordinates, X, Y, and Z.

These parameters consist of an origin shift in three dimensions (Tx Ty Tz), a rotation about each coordinate axis (Rx Ry Rz), and a change in scale (ΔS). The model (Bursa-Wolf model) required for transforming positions from the WGS84 ellipsoid to the Minna datum is given as in Equation 6 as (Featherstone and Vanicek, 1999).

$$\begin{pmatrix} X \\ Y \\ Z \end{pmatrix}_{MINNA} = \begin{pmatrix} T_x \\ T_Y \\ T_Z \end{pmatrix} + (1 + \Delta S) \begin{pmatrix} 1 & R_Z & R_Y \\ -R_Z & 1 & R_X \\ R_Y & R_X & 1 \end{pmatrix} \begin{pmatrix} X \\ Y \\ Z \end{pmatrix}_{WGS84} \tag{6}$$

Where: Where: (φ, λ, h) = the geodetic coordinates of (old ellipsoid), X, Y, and Z.

The Shell Petroleum Development Company engaged in determining refined transformation parameters that enable position determination for crude oil prospecting, exploration, and exploitation in the Niger Delta region of Nigeria using DGPS receivers between the WGS84 and Minna datums. The values are presented in Table 1 (SPDC, 2010).

Table 1: Translation Parameters

Datum shift parameters from WGS 84 to Minna Datum		
	(i) Translation (parameter)	
Dx		Plus 111.916
Dy		Plus 87.852
Dz		Minus 114.499
	(ii) Rotational (paramater)	
Rx		Minus 1.87527″
Ry		Minus 0.20214″
Rz		Minus 0.20214″
Scale factor	Minus 0.03245 ppm	

The Abridged Molodensky Transformation Model for transformation is represented in Equation 7.

$$\begin{bmatrix} \Delta\emptyset \\ \Delta\lambda \\ \Delta h \end{bmatrix} = \begin{Bmatrix} \left[-T_x \sin\emptyset \cos\lambda - T_y \sin\emptyset \sin\lambda + T_z \cos\emptyset\,(a\delta f + f\delta a)\sin 2\theta\right] \big/ (R_M \sin 1'') \\ \left[-T_x \sin\lambda + T_y \cos\lambda\right] \big/ (R_N \cos\emptyset \sin 1'') \\ T_x \cos\emptyset \cos\lambda + T_y \cos\emptyset \sin\lambda + T_z \sin\emptyset + (a\delta f + f\delta a) sin^2\emptyset - \Delta a \end{Bmatrix} \tag{7}$$

$$\begin{Bmatrix} R_M = \frac{a(1-e^2)}{(1-e^2 sin^2\emptyset)^{\frac{3}{2}}} \\ R_N = \frac{a}{(1-e^2 sin^2\emptyset)^{\frac{1}{2}}} \end{Bmatrix}, e^2 = 2f - f^2 \tag{8}$$

ΔØ,Δλ, Δh = correction to transform, (Tx Ty Tz), are the translation parameters, a rotation about each coordinate axis (φ λ h), δa, δf = difference between semi-major axis (a) and the flattening (f), R_M, R_N = radii of curvature in the prime vertical and meridian

Conversion of the Cartesian Rectangular Coordinates on the Local Datum to Geodetic Coordinates (φ, λ, h) on the Local Datum/Ellipsoid.

Having obtained the Cartesian coordinates on the Minna datum, they still need to be converted to curvilinear/geodetic positions on the Minna datum before they can be converted to plane rectangular coordinates such as NTM and UTM coordinates. The Equations required to convert the local datum Cartesian coordinates to curvilinear coordinates are given as in Equation 9 (Janssen, 2009):

$$\begin{aligned} \varphi_{Minna} &= tan^{-1}\left[\frac{z}{\sqrt{X^2+Y^2}}\left(1-e^2\left(\frac{N}{N+h}\right)\right)^1\right] \\ \lambda_{Minna} &= tan^{-1}\left[\frac{X}{Y}\right] \\ h_{Minna} &= \sqrt{X^2+Y^2}\sec\varphi - N \end{aligned} \qquad (9)$$

Where, $e' =$ eccentricity squared $= 2f - f^2$

$N =$ radius of curvature as given in Equation (7).

Conversion of the Geodetic Coordinates (φ, λ, h) to Plane Rectangular Systems

To obtain the positions of points in local plane rectangular systems, the local ellipsoid curvilinear coordinates have to be converted to either NTM or UTM. The models and procedure for conversion of the local ellipsoid geodetic coordinates to either of the two local plane rectangular (NTM or UTM) coordinates are the same. The difference between the two plane systems is in the properties to be used in the conversion. Thus, the origin and scale factors. To convert the geographic coordinates (latitude and longitude) on the local ellipsoid to either NTM or UTM Northing and Easting, using equations (8) given by (Idowu, 2012; Eteje et al.2019; Manchuk, 2009).

$$\begin{Bmatrix} E = & k_0N[A+(1-T+C)A^3/6+(5-18T+T^2+72C-58e'^2)A^5/120] \\ N = & k_0[M-M_0+N\tan Ø[A^2/2+(5-T+9C+4C^2)A^4/24+61-58T+T^2+600C-330e'^2)A^6/720] \\ k = & k_0[1+(1-C)A^2/2+(5-4T+42C+13C^2-28e'^2)A^4/24+(61-148T+16T^2)A^6/720] \end{Bmatrix} \qquad (10)$$

Where:

k_0=0.99975 *for NTM* and 0.9996 *for UTM*

$$e'^2 = \frac{e^2}{1-e^2} = second eccentricity\ squred$$

$$e^2 = 2f - f^2 = eccentricity\ squred$$

$$N = redius\ of\ curvature\ as\ contained\ in\ equation\ 10$$

$$T = tan^2\varphi)$$

$$C = e'^2 cos^2\varphi$$

$$A = (\lambda - \lambda_0)\cos\varphi$$

N = Northing of point. *E* = Easting of point.

φ = latitude of the point

λ = longitude of the point

λo = longitude of the point of the centre meridian of the belt or zone

$$M = a\begin{bmatrix}(1 - e^2/4 - 3e^4/64 - 5e^6/256 - \cdots)\varphi - (3e^2/8 + 3e^4/32 + 45e^6/1024 + \cdots) \\ sin^2\varphi + (15e^4/256 + 45e^6/1024 + \cdots)sin^4\varphi - 35e^6/3072 + \cdots)sin^6\varphi + \cdots\end{bmatrix}$$

M = Distance on the meridian from the parallel of false origin (4°N for NTM and 0° for UTM) to the parallel of the point.

φ = Latitude of the point.

Mo is computed using Equation (9), which is the latitude crossing the central meridian at the origin of the (E, N) coordinates (Ehigiator et al., 2011).

Equations (1) to (9) are used to develop programs in which the transformation/conversion (GNSS) post-processing) software normally applies during computation/conversion or post-processing of static DGPS observations.

For civil engineering projects, it is important to work with plane coordinates, according to Correa-Muños and Cerón-Calderón (2018), because the geometric parameters, like lengths, are based on Euclidean distances. This informs that the ellipsoidal coordinates need to be converted into plane coordinates.

CORS_Geosyatems Performance Testing

In order to decide on the fate of acceptance or rejection of the present observations, the reliability of the CORS_Geosystems was tested (put to the test) by using it to check the existing Raph_UB_02 control point data against the current value.) The validation was to determine the minimum allowable error that could not act as the mitigating factor in the (process of) determination of other unknown monumented controls. Statistical analysis of (2D) position (planimetry) and altimetry (1D) or the integration of both (3D) relative to a known reference control was suggested by (Ariza-López et al., 2021; Joint Committee for Guides in Metrology, 2012) for GNSS observations. If, for instance, x and y have independent random errors δx and δy, then the error in z = x + y. The magnitude of error at each station of observation can be calculated based on the distance from the origin of the survey using equation 10 (Pezzullo, 2016; Taylor, 1997).

$$\delta z = \sqrt{\delta x^2 + \delta y^2} \tag{12}$$

This method accounts for the propagation of error with distance. By adopting Equation 12, the correction to observations between the reference station (CORS_Geosystem) and the known control point (Raph_uB_02) can be computed.

$$C_{r_i} = {}^{d_i}/_{L} \times E_c \tag{13}$$

Where: (C_r)_i denotes the correction applied to station i, d_i is the distance to station i from the CORS_Geosystems, L is the total length of the line between the CORS_Geosystems and Raph_UB_02, and E_c is the misclosure (error)

The relative accuracy of horizontal distance measurements can be expressed as (Pezzullo, 2016; Taylor, 1997):

$$\delta z = \frac{\sqrt{\delta x^2 + \delta y^2}}{L} \tag{14}$$

Adjusted x and y coordinates of Raph_UB_02 is therefore calculated as:

$$C_{rdx_i} = \delta x \times {}^{L_i}/_{L} \tag{15}$$

$$C_{rdy_i} = \delta y \times {}^{L_{ii}}/_{L} \tag{16}$$

where C_{rdx_i} and C_{rdy_i} are the adjustments in x and y coordinates at station *i*

Table 2 shows the summary of the result of the preliminary check on existing control Raph_UB_02 with CORS_Geosystems. The discrepancies in Eastings, Northings and Heights are included.

Table 2: CORS Performance Test Result

S/No	**Contl _ID**	**Obtained**			**Computed**			**Differences**				**Remark**
		Easting (m)	Northig (m)	Height (m)	Easting (m)	Northig (m)	Height (m)	ΔE (m)	ΔN (m)	ΔH (m)	Ellip. Dist. (m)	
1	CORS	791897.989	700539.954	83.291	-	-	-	-	-	-	8137.237	Fixed
2	Raph_ UB_02	789884.712	708294.453	121.023	789884.600	708294.400	122.174	0.112	0.053	-1.151		Ok.
3	Horizontal relative accuracy							0.0000152272		15.2272ppm		Ok

Data Acquisition

Normally, the CORS_Geosystems is the virtual reference station where all observations taken on the newly established controls within the University of Benin will have their relative position tied. Observations time for each occupation took at least 1 hour, with 8 rover receivers simultaneously deployed on each control. Figure 3 shows the picture of the process of data acquisition in one of the control stations within the Ugbowo campus.

Figure 3: Data Acquisition in Progress

Data Post-Processing

The post-processing of the acquired data was done with (the aid of) Trimble Business Centre software. The network was successfully adjusted at 2 iterations using the Trimble Business centre software. The Chi-Square (x^2) result of the adjusted coordinates passed at (95%) precision confidence level with the degree of freedom as one-hundred and twenty (120). The post-processed vector statistics reference factor was 1.00, the number of redundancies was 120, and the prior scalar was obtained as 1.19. Figure 4 represents the network adjustment procedure in the Trimble business centre software used.

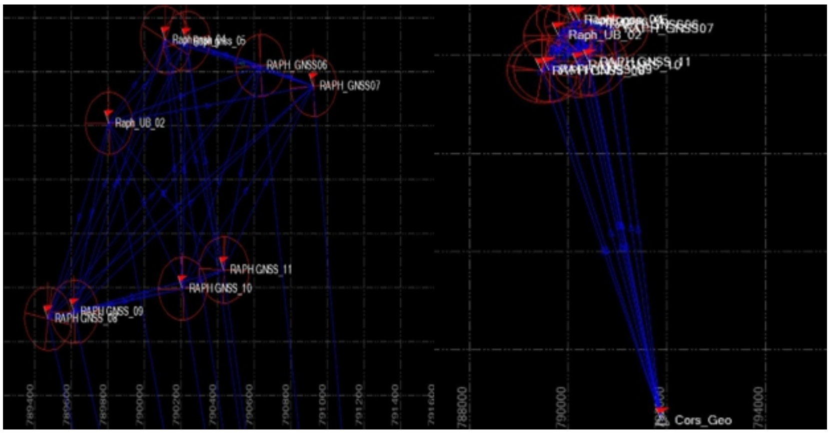


Figure 4: Network Adjustment Procedure

Results and Discussion

The various outputs of the adjusted coordinates in Trimble business centre software are the conversion process results from Minna datum to WGS84. GNSS gives ellipsoidal height, but levelling provides the orthometric height. The difference between the two is referred to as geoidal undulation. Ellipsoidal height will not provide sufficient information to sufficiently model the direction of flow for a small area in a civil engineering project. We used Tersus GNSS NUWA software enhanced with EGM2008 (Geoidal model) to perform the conversion.

Although other results of adjusted GNSS observations, including covariance between the CORS_Geosystems, the existing control, the established controls, and the station interactions both from and back to the CORS_Geosystems as well as the interaction relative to one another in the baseline networks during post-processing are available, the relevant areas have been extracted and presented in Tables 3 to 8 for easy comprehension. Discussions and explanations on each of the Tables and their results are provided immediately after their respective presentations.

Table 3: Adjusted GNSS Observations

Observation ID	Components	Observation	A-posteriori Error	Residual	Standardised Residual
CORS_Geo --> Raph GNSS_05 (PV2)	Az.	348°44'45"	0.040 sec	0.034 sec	0.776
	ΔHt.	4.912 m	0.009 m	-0.028 m	-2.410
	Ellip Dist.	8336.499 m	0.001 m	-0.005 m	-3.282
CORS_Geo --> RAPH GNSS_10 (PV9)	Az.	347°10'44"	0.047 sec	-0.020 sec	-0.546
	ΔHt.	6.415 m	0.009 m	0.033 m	3.042
	Ellip Dist.	7444.385 m	0.002 m	0.002 m	1.490
CORS_Geo --> RAPH GNSS_11 (PV27)	Az.	348°58'45"	0.053 sec	0.058 sec	1.355
	ΔHt.	3.622 m	0.010 m	0.016 m	1.218
	Ellip Dist.	7463.330 m	0.002 m	0.004 m	2.694
CORS_Geo --> RAPH_GNSS06 (PV81)	Az.	351°23'55"	0.044 sec	-0.113 sec	-2.619
	ΔHt.	2.736 m	0.010 m	0.002 m	0.145
	Ellip Dist.	8176.172 m	0.002 m	-0.002 m	-1.382
CORS_Geo --> Raph_UB_02 (PV5)	Az.	345°22'58"	0.041 sec	-0.009 sec	-0.203
	ΔHt.	12.947 m	0.009 m	-0.019 m	-1.607
	Ellip Dist.	8137.237 m	0.001 m	0.002 m	1.543
CORS_Geo --> RAPH_GNSS07 (PV48)	Az.	353°21'08"	0.040 sec	0.010 sec	0.261
	ΔHt.	2.566 m	0.009 m	-0.010 m	-0.892
	Ellip Dist.	8060.947 m	0.001 m	-0.002 m	-1.169
CORS_Geo --> RAPH GNSS_08 (PV14)	Az.	341°31'11"	0.049 sec	0.039 sec	0.674
	ΔHt.	16.332 m	0.011 m	0.009 m	0.574
	Ellip Dist.	7537.808 m	0.002 m	0.000 m	-0.079
CORS_Geo --> RAPH GNSS_09 (PV20)	Az.	342°36'13"	0.049 sec	0.016 sec	0.262
	ΔHt.	14.761 m	0.010 m	0.002 m	0.090
	Ellip Dist.	7517.880 m	0.002 m	-0.001 m	-0.510
CORS_Geo --> Raph GNSS_04 (PV1)	Az.	347°59'24"	0.046 sec	-0.016 sec	-0.417
	ΔHt.	6.656 m	0.010 m	0.001 m	0.057
	Ellip Dist.	8366.055 m	0.002 m	0.001 m	0.412

Table 3 shows the adjusted GNSS observations between the reference CORS and the participating controls. It contains information on the observation ID, the components, the observation, the a-posteriori error, the residual error, and the standardised residual error, respectively. Station Cors_Geo --> Raph GNSS 04 (PV1) has the farthest ellipsoidal distance of 8366.055 m from the reference station, while Cors_Geo --> RAPH GNSS_10 (PV9) station has the least ellipsoidal distance of 7444.385 m. a-posteriori error spread between 0.001m and 0.002 m.

Table 4: The Covariance Terms Between CORS_Geosyatems and Other Stations

From Point	**To Point**		**Components**	**A-posteriori Error**	**Horiz. Precision (Ratio)**	**3D Precision (Ratio)**
CORS_Geo	Raph GNSS_04	Az.	347°59'24"	0.046 sec	1 : 4,712,663	1 : 4,715,307
		ΔHt.	6.656 m	0.010 m		
		ΔElev.	6.234 m	0.010 m		
		Ellip Dist.	8366.055 m	0.002 m		
CORS_Geo	Raph GNSS_05	Az.	348°44'45"	0.040 sec	1 : 6,038,657	1 : 6,035,931
		ΔHt.	4.912 m	0.009 m		
		ΔElev.	4.492 m	0.009 m		
		Ellip Dist.	8336.499 m	0.001 m		
CORS_Geo	RAPH GNSS_08	Az.	341°31'11"	0.049 sec	1 : 4,432,022	1 : 4,432,363
		ΔHt.	16.332 m	0.011 m		
		ΔElev.	15.959 m	0.011 m		
		Ellip Dist.	7537.808 m	0.002 m		
CORS_Geo	RAPH GNSS_09	Az.	342°36'13"	0.049 sec	1 : 4,599,213	1 : 4,597,370
		ΔHt.	14.761 m	0.010 m		
		ΔElev.	14.388 m	0.010 m		
		Ellip Dist.	7517.880 m	0.002 m		
CORS_Geo	RAPH GNSS_10	Az.	347°10'44"	0.047 sec	1 : 4,923,023	1 : 4,922,175
		ΔHt.	6.415 m	0.009 m		
		ΔElev.	6.042 m	0.009 m		
		Ellip Dist.	7444.385 m	0.002 m		
CORS_Geo	RAPH GNSS_11	Az.	348°58'45"	0.053 sec	1 : 4,255,886	1 : 4,256,390
		ΔHt.	3.622 m	0.010 m		
		ΔElev.	3.247 m	0.010 m		
		Ellip Dist.	7463.330 m	0.002 m		
CORS_Geo	RAPH_GNSS06	Az.	351°23'55"	0.044 sec	1 : 5,378,060	1 : 5,377,724
		ΔHt.	2.736 m	0.010 m		
		ΔElev.	2.323 m	0.010 m		
		Ellip Dist.	8176.172 m	0.002 m		
Cors_Geo	RAPH_GNSS07	Az.	353°21'08"	0.040 sec	1 : 5,715,755	1 : 5,715,000
		ΔHt.	2.566 m	0.009 m		
		ΔElev.	2.158 m	0.009 m		
		Ellip Dist.	8060.947 m	0.001 m		
Cors_Geo	Raph_UB_02	Az.	345°22'58"	0.041 sec	1 : 5,582,937	1 : 5,580,134

Table 4 shows the from-point to-point, the components, the a-posteriori error, the horizontal precision, and the 3D precision. The accuracy standard for First-Order control recommended by SURCON (2007) is 1:100,000. The precision obtained from the CORS_Geosystems to the control station showed a better accuracy standard above the recommended 1:100,000, indicating that the GNSS equipment deployed for this study has a high accuracy standard. The maximum 2D and 3D precisions were obtained from CORS_Geo to Raph GNSS_05 as 1:6,038,657 and 1:6,035,931, while the minimum was attained from CORD_Geo to RAPH GNSS_11 as 1:4,255,886 and 1:4,256,390 respectively.

Table 5: Adjusted Grid Coordinates Result

Point ID	Easting (m)	Easting Error (m)	Northing (Meter)	Northing Error (m)	Elevation (m)	Elevation Error (m)	Constraint
Cors_GeoSystem	791979.716	0.000	700426.07	0.000	108.163	0.000	LLh
Raph GNSS_04	790196.019	0.002	708605.325	0.002	114.733	0.010	
Raph GNSS_05	790309.969	0.002	708599.165	0.001	112.990	0.009	
RAPH GNSS_08	789552.402	0.002	707567.514	0.002	124.409	0.011	
RAPH GNSS_09	789693.985	0.002	707593.159	0.002	122.840	0.010	
RAPH GNSS_10	790289.661	0.002	707681.041	0.002	114.500	0.009	
RAPH GNSS_11	790514.686	0.002	707749.147	0.002	111.709	0.010	
RAPH_GNSS06	790714.857	0.002	708509.201	0.002	110.820	0.010	
RAPH_GNSS07	791005.079	0.002	708433.175	0.001	110.654	0.009	
Raph_UB_02	789884.712	0.002	708294.453	0.001	121.023	0.009	

Table 5 shows the ten (10) participating controls, of which row one (1) is the CORS, row ten (10) is the Raph_UB_02, which is the existing control, and the other eight (8) are the newly established ones. The adjusted grid coordinates reveal that errors in easting are 0.002 m all through, but errors in northings varied between 0.001 m and 0.002 m. Errors in height varied between 0.009 m to 0.011 m. The errors are minimal and are, therefore, accepted. The minimum and maximum heights obtained for the newly established control, excluding the CORS and the existing one, are 124.409 m RAPH GNSS_08 and 110.654 m RAPH_GNSS07, respectively.

Table 6: WGS84 Adjusted Geodetic Coordinates

Point ID	Latitude	Longitude	Height (m)	Height Error (m)	Constraint
CORS_Geo	N6°19'51.73746"	E5°38'17.82973"	109.626	0.000	LLh
Raph GNSS_04	N6°24'18.11798"	E5°37'21.18151"	116.282	0.010	
Raph GNSS_05	N6°24'17.89867"	E5°37'24.88613"	114.538	0.009	
RAPH GNSS_08	N6°23'44.46252"	E5°37'00.07956"	125.958	0.011	
RAPH GNSS_09	N6°23'45.27337"	E5°37'04.68800"	124.387	0.010	
RAPH GNSS_10	N6°23'48.03372"	E5°37'24.07348"	116.041	0.009	
RAPH GNSS_11	N6°23'50.21204"	E5°37'31.40243"	113.248	0.010	
RAPH_GNSS06	N6°24'14.90471"	E5°37'38.03802"	112.362	0.010	
RAPH_GNSS07	N6°24'12.38321"	E5°37'47.46329"	112.192	0.009	
Raph_UB_02	N6°24'08.05630"	E5°37'11.00637"	122.573	0.009	

Table 6 contains the result of the WGS84 adjusted geodetic coordinates. The location of each control and the reference stations are presented in terms of their latitude and longitude. The variation in heights shows that they have shifted from local to WGS84. The errors in the heights varied between 0.009 m and 0.011 m.

Table 7: Adjusted ECEF Coordinates

Point ID	X (m)	X Error (m)	Y (m)	Y Error (m)	Z (m)	Z Error (m)	3D error (m)	Constraint
Cors_GeoSystems	6308934.864	0.000	622853.021	0.000	698666.792	0.000	0.000	LLh
Raph GNSS_04	6308209.016	0.010	621032.047	0.002	706800.095	0.002	0.010	
Raph GNSS_05	6308196.884	0.009	621145.250	0.002	706793.205	0.002	0.010	
RAPH GNSS_08	6308396.785	0.010	620398.903	0.002	705773.739	0.002	0.011	
RAPH GNSS_09	6308378.608	0.010	620539.423	0.002	705798.318	0.002	0.010	
RAPH GNSS_10	6308302.604	0.009	621130.565	0.002	705881.657	0.002	0.010	
RAPH GNSS_11	6308270.348	0.010	621353.707	0.002	705947.846	0.002	0.010	
RAPH_GNSS06	6308165.324	0.009	621548.266	0.002	706701.564	0.002	0.010	
RAPH_GNSS07	6308145.345	0.009	621837.348	0.002	706624.569	0.002	0.010	
Raph_UB_02	6308280.174	0.009	620724.847	0.002	706493.635	0.002	0.010	

Table 7 shows the 3D (X, Y, and Z) adjusted ECEF coordinates and their corresponding associated errors. CORS was held fixed. The X-errors are between 0.009 m and 0.10 m, and the Y-errors had a uniform value of 0.002 m. The Z-errors maintained a constant value of 0.002 m as well. The 3D error varied from 0.010 to 0.011 m.

Inspection of Class Type

This aspect tests the confidence in allocating class to the GNSS survey based on the results of a successful minimally constrained least squares adjustment by checking if the semi-major axis of each relative standard error ellipse or ellipsoid, i.e. one sigma is less than or equal to the length of the maximum allowable semi-major axis r using Equation 17 (GWI, 2017).

$$r = c\,(d + 0.2) \qquad (17)$$

Where: r = length of maximum allowable semi-major axis in mm, c = an empirically derived factor represented by historically accepted precision for a particular standard of survey (usually 7.5 is used for class "A" that is specifically meant for densification of geodetic control), and d = distance to any station in km.

Table 8: Computation of control classification

From Point	To Point	Semi-major axis (mm)	Azimuth	Dist. (km)	Class "A" allowable limit
Cors_Geo	Raph GNSS_04	2.000	114°	8.366	7.5 (8.366+0.2) = 64.245
Cors_Geo	Raph GNSS_04	2.000	80°	8.336	7.5 (8.336+0.2) = 64.020
Cors_Geo	RAPH GNSS_08	2.000	100°	7.538	7.5 (7.538+0.2) = 58.035
Cors_Geo	RAPH GNSS_09	2.000	89°	7.518	7.5 (7.518+0.2) = 57.885
Cors_Geo	RAPH GNSS_10	2.000	88°	7.444	7.5 (7.444+0.2) = 57.330
Cors_Geo	RAPH GNSS_11	2.000	93°	7.463	7.5 (7.463+0.2) = 57.473
Cors_Geo	RAPH_GNSS06	2.000	86°	8.176	7.5 (8.176+0.2) = 62.820
Cors_Geo	RAPH_GNSS07	2.000	86°	8.061	7.5 (8.061+0.2) = 61.958
Cors_Geo	Raph_UB_02	2.000	89°	8.137	7.5 (8.137+0.2) = 62.528

Table 8 represents the station description, the semi-major axis in meter, the azimuth, the distance in (kilometres), and the allowable limit for class A, which is a survey meant for densification of controls. Since all the computed limits for class A are greater than the semi-major axis values of (2.000 mm). It follows that the survey met the first-order criteria; hence, all the newly established controls are first-order and class "A" compliance (GWI, 2017).

Conclusion

The confirmation of the performance of the CORS_Geosystems was successfully carried out, and the densification of eight other first-order control points was achieved within the University of Benin, Ugbowo campus using a single reference (CORS_Geosystems). These controls are therefore recommended to fit for purpose and useful for engineering projects and other related developmental applications.

Recommendation

It is recommended that the newly established controls be protected and utilised for developmental activities, mapping and various other applications in and around the vicinity of the University.

References

Ariza-López F.J., García-Balboa, J.L., Rodríguez-Avi, J., & Robledo J., (2021). Guide for the Positional Accuracy Assessment of Geospatial Data. Pan American Institute of Geography and History, Occasional Publication # 563

Ayodele, E.G., Okolie C.J. & Mayaki O. A., (2019) An Assessment of the Reliability of the NIGNET Data. *Nigerian Journal of Environmental Sciences and Technology (NIJEST*). *3(1),* pp 18 – 29. ISSN (electronic): 2616-0501.

Correa-Muñoz, N.A., Cerón-Calderón, L.A. (2018). Precision and accuracy of static GNSS for surveying networks used in Civil Engineering. *Ingeniería e Investigación, 38(1),* 52-59. DOI: 10.15446/ing.investig.v38n1.64543 .

Ehigiator, M.O, & Oladosu, S.O. (2017). Determination of Conversion Constant between Lagos Datum and Niger Delta Mean Lower Low Water Datum and their Horizontal and Vertical Accuracy Standards using

GNSS Observations. *Nigerian Journal of Environmental Sciences and Technology (NIJEST) 2(1),* 56 – 68.

Ehigiator, M.O. Oladosu, S.O. & Ehigiator-Irughe R., (2017): "Densification of (GNSS) Control Points for Cadastral and Mapping Purposes" *Nigerian Journal of Environmental Sciences and Technology (NIJEST) 1(2),* 287-298.

Ehigiator-Irughe, R. & Audu, H.A.P., (2016). Determination of Horizontal and vertical Positional Accuracy from GNSS Geodetic Observation. *Tropical Environment.* 13(1): 34-45.

Ehigiator-Irughe, R., Ehigiator, M.O & Uzoekwe, S.A. (2011). Establishment of Geodetic Control in Jebba Dam using CSRS – PPP Processing Software, *Journal of Emerging Trends in Engineering and Applied Sciences* (JETEAS) 2 (5), 763 – 769.

Eteje, S. O., Oduyebo O. F., & Oluyori P. D. (2019). Procedure for Coordinates Conversion between NTM and UTM Systems in Minna Datum Using All Trans and Columbus Software. *International Journal of Scientific Research in Science and Technology, 6(5),* 128-143.

Ezeigbo, C. U., (2004) Integrating Nigerian Geodetic Network into the African Geodetic Reference Framework: What are Issues? In: *Nigerian Institution of Surveyors (NIS) 39th AGM and Conference*, Port-Harcourt, Nigeria.

Featherstone, W. & Vanicek, P. (1999): The Role of Coordinate Systems, Coordinates and Heights In: Horizontal Datum Transformations. The Australian Surveyor, 44(2). In: Ono, M. N. (2009): On Problems of Coordinates, Coordinate Systems and Transformation Parameters in Local Map Production, Updates and Revisions in Nigeria. FIG Working Week, Eilat, Israel.

Geomatics Work Instruction (GWI, 2017). Regulations of Geomatics (Work Instructions) Part II Issue: 1 Revision: 1 (Geodetic) Survey Department Brunei Darussalam. TeamWARE Library\UKUR ISO 9001:2000\Work Instruction\Geomatic WI Doc No: GWI_Part II_Geodetic.

Ghilani, C. D., & Wolf, P. R. (2012). *Elementary Surveying: An Introduction to Geomatics (13th Edition). Vasa.* Prentice Hall

Heiskanen, W. & Moritz, H., (1967). Physical Geodesy. Institute of Physical Geodesy, Technical University, Graz, Australia. Freeman, San Francisco.

Idowu, T. O. (2012). Comparison of Numerical Techniques for Coordinate Transformation: The Case Study of Nigeria Transverse Mercator and Universal Transverse Mercator. *International Journal of Applied Science and Technology*, 2(4). 122-128.

Iyiola F., Ogundele R., Oluwadare C. & Kufoniyi O., (2013). Integrity Check on Ground Control Points using NIGNET's Continuously Operating Reference Stations FIG Working Week 2013 Environment for Sustainability Abuja, Nigeria, 6-10 May 2013

Jatau, B., Fernandes R.M.S., Adebomehin A. & Gonçalves N., (2010).

NIGNET-The New Permanent GNSS Network of Nigeria (4549) GNSS CORS Networks - Infrastructure, Analysis and Applications II. Facing the Challenges. Building the Capacity Sydney, Australia, *FIG Congress* 11-16 April 2010

Janssen, V. (2009). Understanding Coordinate Systems, Datums and Transformations in Australia. In: Ostendorf, B., Baldock, P., Bruce, D., Burdett, M. and P. Corcoran (eds.), *Proceedings of the Surveying & Spatial Sciences Institute Biennial International Conference*, Adelaide 2009, Surveying & Spatial Sciences Institute, 697-715.

Joint Committee for Guides in Metrology (2012). JCGM 200:2012. International vocabulary of metrology – Basic and general concepts and associated terms (VIM), 3rd edition. Working Group 2 of the Joint Committee for Guides in Metrology (JCGM/WG2).

Karaim, M., Elsheikh, M., & Noureldin, A. (2018). GNSS Error Sources. In R. B. Rustamov, & A. M. Hashimov (Eds.), Multifunctional Operation and Application of GPS. IntechOpen. https://doi.org/10.5772/intechopen.75493

Leica Geosystems (2005). GPS Reference Stations and Networks: An introductory guide. Copyright Leica Geosystems AG, Heerbrugg, Switzerland, 2005. 747352en-IV.05-RVA

Manchuk, J. G. (2009). Conversion of Latitude and Longitude to UTM Coordinates. Paper 410: 1-4, CCG Annual Report.

Ojigi, L. M. (2015) Leveraging on GNSS Continuously Operating Reference Stations (CORS) Infrastructure for Network Real Time Kinematic Services In in Nigeria', African Journal of Applied Research (AJAR), 1(1).

Office of the Surveyor General of the Federation (OSGoF, 2012) Report on NIGNET GNSS Data Processing, 2010 - 2011. Office of the Surveyor General of the Federation, Abuja, Nigeria.

Oladosu, S.O, Ehigiator-Irughe, R., & Muhammad, M.B.M.E., (2022) Establishment and Validation of Continuously Operating Reference Stations Geosystems Network on Static and Real-Time Kinematic in Benin City, Nigeria. *J. Appl. Sci. Environ. Manage.* 26 (5) 801-808. DOI: https://dx.doi.org/10.4314/jasem.v26i5.4

Ono M.N. (2009). On Problems of Coordinates, Coordinate Systems and Transformation Parameters in Local Map Production, Updates and Revisions in Nigeria *FIG Working Week 2009*, TS 5C-Geodetic Datum II: Surveyors Key Role in Accelerated Development Eilat, Israel, 3–8 May, 2009

Pezzullo, C. J., (2016). Simple Error Propagation Formulas for Simple Expressions. In: The Book "Biostatistics for Dummies". Retrieved April 20, 2022 from https://www.dummies.com/article/academics-the-arts/science/biology/simple-error-propagation-formulas-for-simple-expressions-149357/

The Shell Petroleum Development Company (SPDC, 2010) Determination of Transformation Parameters for Niger Delta Region of Nigeria.

Surveyors Council of Nigeria (SURCON, 2007) "Specifications for Geodetic Surveys in Nigeria. GPS Surveys Specifications, 44-46. Lagos-Nigeria.

Shirazian M., Jazireeyan, I. & Bagherbandi, M. (2020). Reality Measure of the Published GPS Satellite Ephemeris Uncertainties, *Journal of Spatial Science*, 67(2), 287–295 https://doi.org/10.1080/14498596.2020.1746702.

Taylor, J.R. (1997). An Introduction to Error Analysis the Study of Uncertainty in Physical Measurement. Second edition. University Science Books, Sausalito, California.

Udochukwu, E.S, Ono, M.N. & Ifeanyi, U.C., (2019). Stability Determination of Second (2nd) Order Federal Geodetic Infrastructure in Anambra State, Nigeria. The *International Journal of Engineering and Science (IJES)*, 8(11); 53-59. ISSN (e): 2319-1813 ISSN (p): 23-19-1805. DOI:10.9790/1813-0811025359

Uzodinma, V. N., Oguntuase, J. O., Alohan, & Dimgba, C. N. (2013). Practical GNSS Surveying. Professor's Press Limited, Enugu.

Wolf, P. R. & Ghilani, C. D. (1967). *Adjustment Computations*, John Wiley and Sons, Inc, New York, 1997. ISBN 0-471-16833-5.

Wu Q., Kang, J., Li S., Zhen J., & Li H. (2015). GNSS Positioning by CORS and EGM 2008 in Jilin Province, China. *Sensors (Basel, Switzerland)*, 15(12), 30419–30428. https://doi.org/10.3390/s151229806.