IOSR Journal of Environmental Science, Toxicology and Food Technology (IOSR-JESTFT)
e-ISSN: 2319-2402,p- ISSN: 2319-2399.Volume 12, Issue 6 Ver. I (June. 2018), PP 19-29
www.iosrjournals.org

Comparative Analysis of DGPS and Total Station Accuracies for Static Deformation Monitoring Of Engineering Structures

Ono, M. N., Eteje, S. O. and Oduyebo, F. O.
Department of Surveying and Geoinformatics, Nnamdi Azikiwe University Awka
Corresponding Author: Ono, M. N., Eteje, S. O

Abstract: *The displacements of engineering structures have been investigated by different researchers using one of the various geodetic methods such as GPS, Total Station, Levelling, InSAR, Photogrammetry, Theodolite, EDM and 3D Laser but in that, the accuracy of these methods have not really been compared to determine which of the methods is better in terms of accuracy, magnitude and direction of the expected deformation. This study monitored and modelled the deformation of Palm House in Benin City using GPS and Total Station and compared the accuracy of the two horizontal methods. Four reference stations and two sets of monitoring points were used. The DGPS observations were used to determine the roof monitoring points rectangular coordinates while the total station was used to determine the 10th floor base monitoring points rectangular coordinates and their bearings and distances from the reference stations. The observations were carried out at six epochs of three months interval and adjusted using least squares adjustment technique to determine the reliability of the adjusted observations and that of the adjusted parameters. The displacements magnitudes of the two sets of observations were computed using the coordinates differences between the first and the subsequent epochs observations. The evaluated displacements magnitudes were compared with their corresponding computed 95% confidence ellipses to determine the significance level. The results showed that neither the 10th floor nor the entire building underwent any movement during the monitoring period of eighteen months. The results of comparison using their a posteriori standard errors and traces of the variance co-variance matrices to determine which of them is better in terms of accuracy showed that the DGPS method is better. It was recommended that whenever more suitable and accurate method of monitoring of engineering structures that DGPS method should be selected.*

Keywords: *Deformation, Monitoring, DGPS, Epoch, Coordinates, Accuracy, Analysis*

Date of Submission: 29-05-2018 Date of acceptance: 12-06-2018

I Introduction

Deformation monitoring also referred to as Deformation Survey is the systematic measurement and tracking of the alteration in the shape or dimensions of an object as a result of stresses induced by applied loads (Uzodinma, 2014). Deformation monitoring is to guarantee the structure safety and detect the abnormal changes, make judgment on the stability and safety of the building. It can be required for Bridges and Viaducts, High-rise and historical buildings, dams, Tunnels, Roads, Foundations, Construction sites, Mining, Landslide and Volcanoes Slopes, Settlement areas and Earthquake areas monitoring.

The purpose of deformation monitoring is to determine whether or not movement is taking place and subsequently whether the structure is stable and safe. Movement takes place as a result of changes in the bedrock, increase or decrease of weight, changes of the material properties as a result of changes in temperature, aging or outside influences (Aghedo, 2016).

The measuring devices used for deformation monitoring depend on the application, the chosen method and the required regularity. Deformation measuring devices are grouped into two, the Geodetic measuring devices which involve the use of total stations, levels, global navigation satellite system receivers, 3D Laser Scanner, Photogrammetry and InSAR to measure georeferenced displacements or movements in one, two or three dimensions, and the Geotechnical measuring devices which involve the use of extensometers, piezometers, rain gauges, thermometers, barometers, tilt meters, accelerometers, seismometers to measure non-georeferenced displacements or movements and related environmental effects or conditions.

The total station measures horizontal and vertical angles and slope distances to each prism from which easting, northing and height values and subsequently displacements are computed. Total station coordination monitors deformation of buildings through the variation of coordinates of observed points. This method offers acceptable accuracy without demanding high visibility or heavy workload. The use of total station surveying instruments for monitoring structures movement gives accurate and good results. The application of total station to monitor building stability is still widely used.

GPS techniques have several advantages as a monitoring tool. The surprisingly high accuracy of relative GPS measurements are finding an application in monitoring surveys in areas where stations require intervisibility and weather conditions. Currently, with the deployment of the full satellite constellation, continuous and automated monitoring using GPS will become increasingly practical and cost-effective. Thus, the potentials of GPS as a super positioning tool brought a fresh air to the field of monitoring surveys, especially in areas where quick results could save lives and property (Aziz et al, 2001)**.** In principle, the monitoring of high-rise building using GPS can be performed episodically (epoch intervals) or continuously.

The displacements of engineering structures have been investigated by different researchers using one of the various geodetic methods such as GPS, Total Station, Levelling, InSAR, Photogrammetry, Theodolite, 3D Laser, etc, but the accuracy of these methods have not really been compared to determine which of the methods is better in terms of their accuracy. The selection of the method of measurements depends upon the accuracy requirements. This in turn is determined by the purpose of measurements and by the magnitude and direction of the expected deformation or movements (Abdel-Gawad et al, 2014).

The aim of the study is to monitor Palm House (high rise building) in Benin City by carrying out some survey observations (DGPS and total station observations) and rigorous analysis with a view of detecting any horizontal displacement of the building and comparing the accuracy of the two horizontal methods for engineering structures displacements monitoring. Its objectives are:

1. To carry out observations (DGPS and Total Station observations) at 6 different epochs at interval of three months and processing of the observations to determine the coordinates and heights of the monitoring networks points.
2. To carry out least squares adjustment and statistical analysis on the observations to determine the reliability as well as the precision, accuracy and uncertainty of the adjusted observations and those of the adjusted parameters.
3. To determine the magnitudes and confidence ellipses at 95% confidence level of the horizontal displacements of the monitoring points and comparing the determined displacements magnitudes with their corresponding confidence ellipses to determine if the displacements are significant or not.
4. To determine and differentiate the movement of the 10th floor from that of the entire building by comparing the results obtained from the two horizontal (GPS and total station) methods.
5. To compare the results obtained from the two horizontal (GPS and total station) methods so as to determine which of the two horizontal methods is better in terms of accuracy.

The monitored structure was Palm House (Figure 1). It is one of the Edo State Secretariat buildings in Benin City. It is a high rise building located along Benin Sapele road in Oredo Local Government Area of Edo State. The building is 45m in length, 15m in breadth and 35m in height. It is an eleven story building. The study area lies between latitudes 06^0 18' 38''N and 06^0 19' 29''N and longitudes 05^0 37' 24''E and 05^0 37' 38''E. Figures 2a to c show the maps of the study area.

***Fig. 1:** Palm House, Benin City*

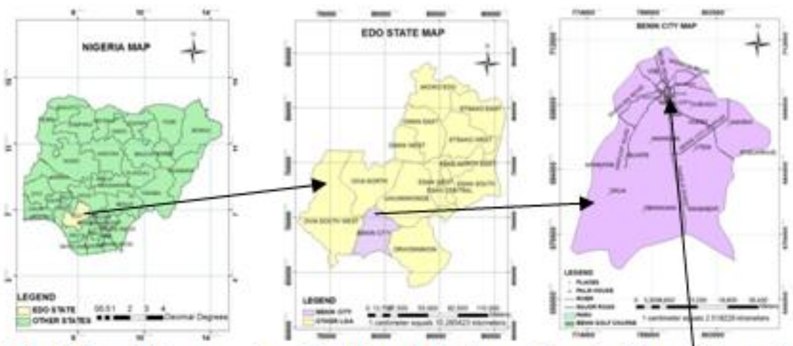

***Fig. 2a:** Map of Nigeria showing Edo State.* *Fig. 2b: Map of Edo State Showing Benin City* *Fig. 2c: Map of Benin City Showing Study Area*

Source: Ministry of Lands and Surveys, Benin City **Study Area**

Least Squares Adjustments by Observation Equation Method

The functional relationship between adjusted observations and the adjusted parameters is given as (Ono et al, 2014):

$$L_a = F\ (X_a) \qquad (1)$$

Where, L_a = adjusted vector of observations and X_a = adjusted station coordinates. Equation (1) is linear function and the general observation equation model was obtained.

To make the matrix expression for performing least squares adjustment, analogy will be made with the systematic procedures. The system of observation equations is presented by matrix notation as (Mishima and Endo 2002):

$$V = AX - L \tag{2}$$

where,

A = Design Matrix, X = Vector of Unknowns, L = Calculated Values (lo) Minus Observed Values (l_b), V = Residual Matrix

That is,

$$\begin{array}{cccc} V & A & X & L \\ \begin{pmatrix} v_1 \\ v_2 \\ \cdots \\ v_m \end{pmatrix} = & \begin{pmatrix} a_{11} & a_{12} & \cdots & a_{1n} \\ a_{21} & a_{22} & \cdots & a_{2n} \\ \cdots & \cdots & \cdots & \cdots \\ a_{m1} & a_{m2} & \cdots & a_{mn} \end{pmatrix} & \begin{pmatrix} x_1 \\ x_2 \\ \cdots \\ x_n \end{pmatrix} - & \begin{pmatrix} l_1 \\ l_2 \\ \cdots \\ l_m \end{pmatrix} \end{array} \tag{3}$$

Estimated parameter

$$X = (N)^{-1}(t) \tag{4}$$

Where, $N = (A^T WA) =$ Normal Matrix, $t = (A^T WL)$, $N^{-1} = (A^T WA)^{-1} = Q_{XX}$

$X = (A^T WA)^{-1}(A^T WL)$ = Estimate, W = Weight Matrix

The models for the computation of the a posteriori variance and a posteriori standard error as given in Ameh (2013) are:

$$\text{A Posteriori Variance } (\hat{\sigma}_o^2) = \frac{V^T WV}{m-n} = \frac{V^T WV}{r} \tag{5}$$

$$\text{A Posteriori Standard Error } (\hat{\sigma}_o) = \sqrt{\frac{V^T WV}{m-n}} = \sqrt{\frac{V^T WV}{r}} \tag{6}$$

Where, m - n = r = Degree of freedom

The model for the computation of the standard error of the adjusted parameters is given as: (Ameh, 2013):

$$\hat{\sigma}_{xi} = \hat{\sigma}_\circ \sqrt{Q_{nn}} = \sqrt{\hat{\sigma}_o^2 Q_{nn}} \tag{7}$$

Where, Q_{nn} is a diagonal element of the inverse of the normal matrix (N^{-1}).

The semi-major axis $\sigma_{x^I}^2$, semi-minor axis $\sigma_{y^I}^2$ and the orientation of the error ellipse θ as given in Mikhail and Gracie (1981) are:

$$\sigma_{x^I}^2 = \frac{\sigma_x^2 + \sigma_y^2}{2} + \left[\frac{(\sigma_x^2 - \sigma_y^2)^2}{4}\right] + \sigma_{xy}^2, \quad \sigma_{y^I}^2 = \frac{\sigma_x^2 + \sigma_y^2}{2} - \left[\frac{(\sigma_x^2 - \sigma_y^2)^2}{4}\right] + \sigma_{xy}^2 \tag{8}$$

$$\tan 2\theta = \frac{2\sigma_{xy}}{\sigma_x^2 - \sigma_y^2} \tag{9}$$

The trace of a square matrix A is the sum of the diagonal elements of A, written as tr A or sometimes tr (A), that is,

$$\text{tr A} = \sum a_{ii} \tag{10}$$

If A is a covariance matrix, then tr A is the sum of all variances and can be interpreted as a measure of the overall accuracy of the associated vector of random variates (Caspary, 1988).

The redundancy number of each adjusted observation (r_i) and average redundancy number of a group of adjusted observations (r_{Av}) are respectively given in (Leick, 1990) as:

$$r_i = q_i p_i \tag{11}$$

$$0 \le r_i \le 1$$

$$r_{Av} = \frac{n - R(A)}{n} \tag{12}$$

Where, q_i = Diagonal element of the estimated residual cofactor matrix, Q_v, p_i = Weight of the i-th observation. $n - R(A)$ = Degree of freedom or redundant observation, $R(A)$ = Number of unknown parameter, n = Number of observation.

Computation of Magnitude and Direction of Displacement

Once the adjustment of observations is completed, object point displacement is computed as the difference in coordinates between the measurement epochs as given in Ehiorobo and Ehigiator, (2011) as:

$$\begin{Bmatrix} x_j^{k+1} - x_i^k = dx \\ y_j^{k+1} - y_i^k = dy \\ z_j^{k+1} - z_i^k = dz \end{Bmatrix} \tag{13}$$

where,

$x_j^{k+1}, y_j^{k+1}, z_j^{k+1}$ = coordinates of last epoch

x_i^k, y_i^k, z_i^k = coordinates of preceding epoch

Horizontal movement (ds) is computed for each object (monitoring) point as:

$$ds = \sqrt{(dx)^2 + (dy)^2} \tag{14}$$

and the direction of movement (α) is computed as:

$$\tan \alpha_i = \frac{dy_i}{dx_i} \tag{15}$$

Deformation Analysis

Deformation modelling or analysis is done to determine whether point displacements are significant (Bird, 2009). To determine the significant of points displacements, the computed displacements are compared with their corresponding 95% confidence ellipses.

If the magnitude of the displacement of a point *j* is classified D_j and the maximum dimension of combined 95% confidence ellipse for point *j* is designated E_j, then, if $\left|D_j\right| < E_j$ we conclude that no movement has occurred in point j but rather the difference observed is as a result of measurement error. But if on the other hand $\left|D_j\right| > E_j$ then we conclude that point movement has occurred (Ehiorobo and Ehigiator, 2011). $\left|D_j\right|$ and E_j are computed as:

$$\left|D_j\right| = \sqrt{(\Delta x_j)^2 + (\Delta y_j)^2} \tag{16}$$

$$E_j = 1.96\sqrt{(m_{\Delta j}^{k+1})^2 + (m_{\Delta j}^k)^2} = 1.96\sqrt{M} \tag{17}$$

where,

$$M = (m_{\Delta j}^{k+1})^2 + (m_{\Delta j}^k)^2$$

$M = (m_{\Delta j}^{k+1})$ - standard error in position for the K+1 epoch and

$= (m_{\Delta j}^k)$ - standard error in position for the previous epoch k.

II Methodology

The adopted methodology involved the following stages, namely: data acquisition, data processing, data analysis and presentation. Figure 3 shows the flow chart of the adopted methodology.

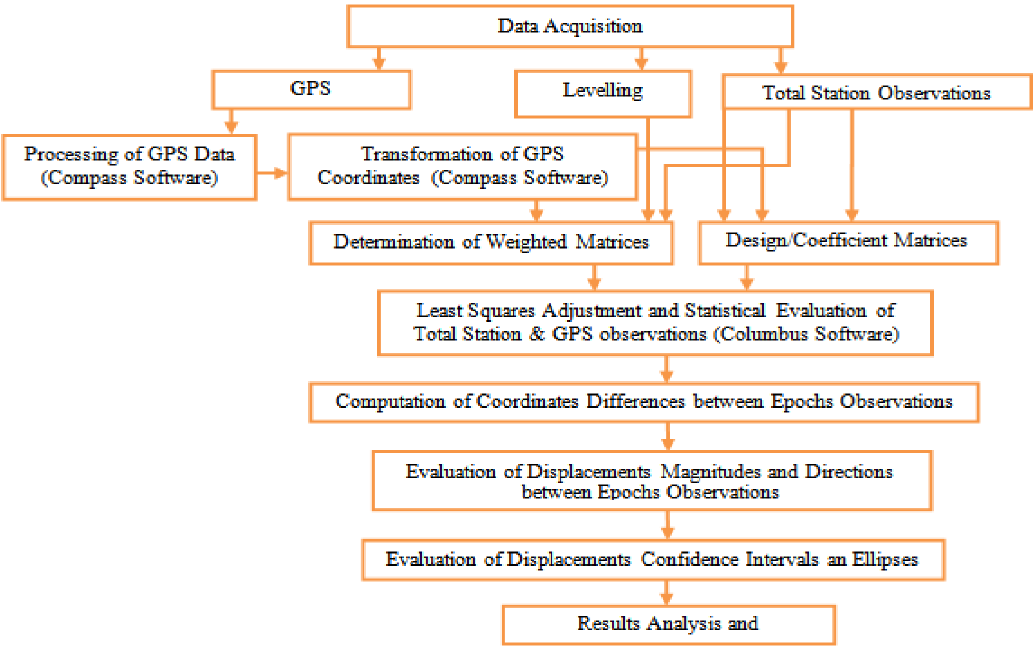

Fig. 3: Flowchart of the Adopted Methodology

2.1 Method of Data Acquisition

Reconnaissance and Monumentation

Prior to the GPS and total station observations, the study area was visited to choose suitable points for reference stations on stable grounds or platforms outside the building and to mark out monitoring points on the building. Two sets of monitoring points were marked on the building: the 10th floor monitoring points for total station observation and the roof monitoring points for DGPS observations. The marked monitoring points were made permanent by driving in concrete nails at the marked points. Nearby controls were also located and their coordinates were obtained from the Edo State Ministry of Lands and Surveys, Benin City. Pre-cast Property Beacons with dimensions 35cm x 35cm x75cm were emplaced at the four chosen reference points. Each of the beacons was capped with a mixture of cement, sand and water. During this process, the centre of each of the beacons was marked and a number template was engraved on them.

Data Acquisition

The rectangular coordinates of the reference stations (A, B, C and D) were determined relative to a nearby control station (FGPEDY06) with DGPS (Figure 4) The DGPS was also used to determine the rectangular coordinates of the roof monitoring points from two different reference stations (B and C) (Figures 5 and 6) in each epoch. The total station was used to determine the bearings, distances and rectangular coordinates of the 10th floor monitoring points (Figure 7). There were six epochs of observations. The observations were taken at three months interval. The first to the sixth epoch observations were respectively taken in March, 2016; June, 2016; September, 2016; December, 2016; March, 2017 and June, 2017. All the observations were taken in the morning hours. Figures 8 and 9 respectively show the DGPS and the Total Station observations networks.

Fig. 4.: Base receiver at Control Station FGPEDY06

Fig. 5: Base receiver at Reference Station C

Fig. 6: Rover receiver at Monitoring point M

Fig. 7: Total Station at Reference Station C

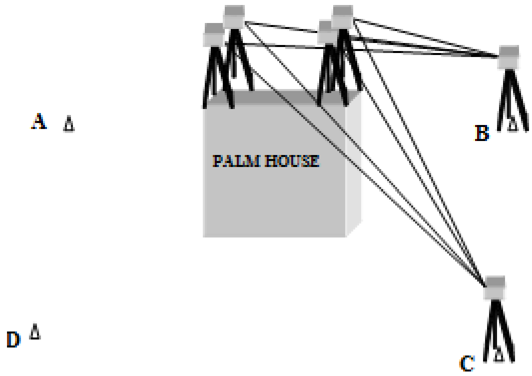

Fig. 8: DGPS Observation Network

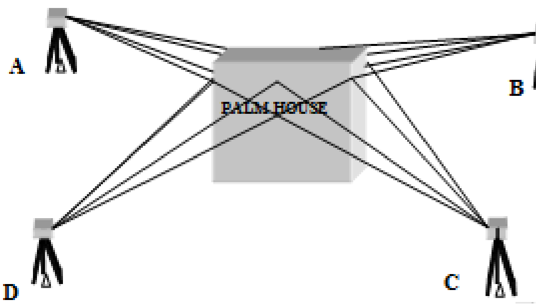

Fig. 9: Total Station Observation Network

2.2 Data Processing Procedure

Processing of the GNSS Observations and the Total Station Data

The DGPS observations were downloaded into a computer system folder using HcLoader software. The downloaded data were processed and transformed to Minna datum coordinates using Compass software. The total station data (bearings, distances, northings and eastings) were typed in Microsoft Excel 2007 spreadsheet and saved in a computer folder.

Least Squares Adjustment of the Processed Coordinates

The least squares adjustment of the total station and the DGPS observations were carried out using Columbus software to determine the reliability of the adjusted observations and those of the adjusted parameters. The DGPS and the total station observations were adjusted using equations (3) and (4). The design matrix (A), observation matrix (L), residual matrix (V), matrix of unknown (X) and weight matrix (W) of the adjusted DGPS observations were respectively 32 x 8, 32 x 1, 32 x 1, 8 x 1 and 32 x 32 matrices while those of the total station observations (distances and azimuths) were respectively 16 x 12, 16 x 1, 16 x 1, 12 x 1 and 16 x 16 matrices.

Statistical Evaluations

The a posteriori variances, a posteriori standard errors, and standard errors of the adjusted coordinates, redundancy numbers, standardized residuals, standardized residual rejection constants (tau statistics), confidence regions (error ellipses) and traces of the adjusted observations and positions were evaluated using Columbus 3.8 software. The a posteriori variances and a posteriori standard errors of the observations were respectively determined using equations (5) and (6) while the standard errors of the adjusted coordinates were determined using equations (7). The semi-major axis $\sigma_{x'}^2$, semi-minor axis $\sigma_{y'}^2$ and the orientation of the error ellipses were respectively computed using equations (8) and (9). The computed semi-major $\sigma_{x'}^2$ and semi-minor $\sigma_{y'}^2$ axes were scaled at 95% confidence level. The traces, redundancy numbers and the average redundancy numbers of the observations were respectively computed using equations (10), (11) and (12).

Deduction of the Displacements of the Building

The first epoch observations of the DGPS and the total station were used as reference observations such that the subsequent epochs observations were compared with them which implies that there were three, six, nine twelve and fifteen months intervals. The displacement magnitude of each monitoring point was determined using equations (13) and (14). The direction of movement was not computed as there was no movement.

Evaluation of Displacements Confidence Ellipses

The confidence ellipses of the displacements of the building were determined with the standard errors of the adjusted coordinates at 95% confidence level using equation (17). The computed displacements magnitudes were compared with their corresponding confidence ellipses to determine if the computed displacements were significant or not.

2.3 Results Analysis

Analysis of the Total Station Observation Results

The total station observation was carried out in six different epochs to determine any movement of the entire building. The total station observations results were seen to be in good shape and were accepted as the coordinates of each monitoring point were able to be reproduced from not less than three reference stations.

Analysis of the DGPS Observation Results

The DGPS observation was carried out in six different epochs to determine and differentiate any movement of the 10th floor from that of the entire building. The processing results of the six epochs DGPS observations were seen to have passed the Network Adjustment Test which implies that the normal matrix generated was a regular one and inverted accordingly for calculation of residuals.

Analysis of the Adjusted Total Station Observations Results Using Least Squares Technique

The a posteriori variances of the first to the sixth epoch total station observations were respectively 0.17488m, 0.21221m, 0.20672m, 0.18981m, 16591m and 0.22807m while their respective a posteriori standard errors were 0.41819m, 0.46066m, 0.45467m, 0.43568m, 0.40733m and 0.47757m which respectively show the high precision and accuracy of the adjusted observations. The maximum standard errors in northing and easting of the first to the sixth epoch adjusted coordinates were respectively 0.00064m and 0.00069m, 0.00068m and 0.00057m, 0.00067m and 0.00056m, 0.00065m and 0.00054m, 0.00060m and 0.00051m, and 0.00071 and 0.00059. These show the high accuracy of each of the adjusted coordinates. The minimum redundancy number of the adjusted distances and azimuths were respectively 0.9999 and 0.0001 while the average redundancy number was 0.625. Though the minimum azimuths redundancy number was close to zero but the observations were accepted as the average redundancy number was 0.625 which also show high precision of the adjusted observations. The computed standardized residuals of the six epochs of observations were all less than their respective standardized residual rejection constant which implies that there were no gross errors or outliers hence, none of the observations was rejected. The maximum scaled semi-major axis and semi-minor axis of the six epochs adjusted coordinates were respectively 0.0019242m and 0.001689m which also show the high accuracy of the adjusted positions. The traces of the first to the sixth epoch observations were respectively 0.0000035148m, 0.000002485m, 0.00000242m, 0.000002222m, 0.000001942m and 0.00000267m which show the high accuracy of each of the adjusted epochs observations.

Analysis of the Adjusted DGPS Observations Results Using Least Squares Technique

The a posteriori variances of the first to the sixth epoch total station observations were respectively 0.0000044705m, 0.0000003131m, 0.0000002144m, 0.0000060425m, 0.0000043369m and 0.0000228554m while their respective a posteriori standard errors were 0.002114344m, 0.000559521m, 0.000463080m, 0.002458150m, 0.002082528m and 0.004780734m which respectively show the high precision and accuracy of the adjusted observations. The maximum standard errors in northing and easting of the first to the sixth epoch adjusted coordinates were respectively 0.00045m and 0.00068m, 0.00027m and 0.00025m, 0.00019m and 0.00019m, 0.00032m and 0.00026m, 0.00029m and 0.00024m, and 0.00045m and 0.00029m. These show the high accuracy of each of the adjusted coordinates. The minimum redundancy number of the adjusted change in northing, change in easting and the average redundancy numbers was 0.75 which is close to 1 showing the high precision of the adjusted observations. The computed standardized residuals of the six epochs observations were all less than their respective standardized residual rejection constant which implies that there were no gross errors or outliers hence, none of the observations was rejected. The maximum scaled semi-major axis and semi-minor axis of the six epochs adjusted coordinates were respectively 0.001102m and 0.001665m which also show the high accuracy of the adjusted positions. The traces of the first to the sixth epoch observations were respectively 0.0000019786m, 0.000000392m, 0.000000223m, 0.000000425m, 0.000000308m and 0.00000051m which show the high accuracy of each of the adjusted epoch observations.

Comparison between the Total Station Observations Displacements Magnitudes and their Corresponding 95% Confidence Ellipses/Regions

The horizontal displacements (total station observations) magnitudes of the building at three, six, nine, twelve and fifteen months intervals were compared with their corresponding 95% confidence ellipses to determine if the computed movements were statistically significant or not (Table 1 and Figure 10). It can be respectively seen from table 1 and figure 10 that the evaluated displacements magnitudes were all less than their corresponding confidence ellipses implying that the building did not undergo any horizontal displacement during the period of observations regarding Ehiorobo and Ehigiator (2011)

Table 1: Comparison of the Horizontal (Total Station Observations) Displacements Magnitudes with their Corresponding Confidence Ellipses

MONITORING POINT		P	Q	R	S	T	U
B/W 1^{ST} & 2^{nd} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000282	0.000400	0.000400	0.000376	0.000714	0.000339
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000542	0.000926	0.000576	0.000625	0.00088	0.000715
	DIFFERENCE (m)	0.000260	0.000527	0.000176	0.000249	0.000166	0.000376
B/W 1^{ST} & 3^{rd} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000449	0.000777	0.000400	0.000389	0.000446	0.000362
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000557	0.000909	0.000604	0.000625	0.000865	0.000738
	DIFFERENCE (m)	0.000108	0.000132	0.000204	0.000236	0.000419	0.000376
B/W 1^{ST} & 4^{th} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000449	0.000769	0.000581	0.000352	0.000714	0.000000
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000585	0.000877	0.000631	0.00068	0.000819	0.000761
	DIFFERENCE (m)	0.000135	0.000108	0.000051	0.000327	0.000104	0.000761
B/W 1^{ST} & 5^{th} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000114	0.000777	0.000581	0.00053	0.000707	0.000383
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000655	0.000794	0.000698	0.000734	0.000783	0.000813
	DIFFERENCE (m)	0.000541	0.000017	0.000117	0.000204	0.000075	0.00043
B/W 1^{ST} & 6^{th} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000273	0.000354	0.000400	0.000304	0.000412	0.000469
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000514	0.000983	0.000549	0.000588	0.000912	0.000693
	DIFFERENCE (m)	0.000242	0.000629	0.000149	0.000284	0.000500	0.000224

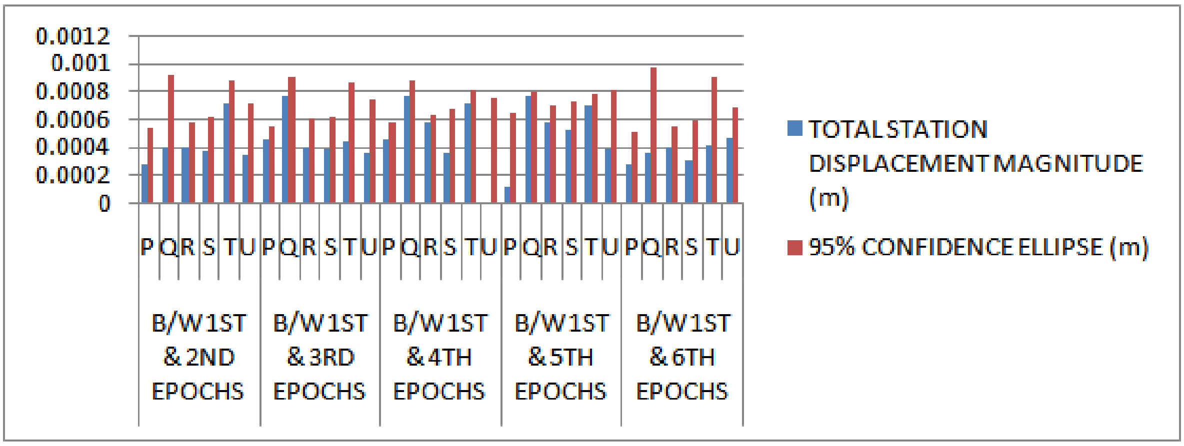

Fig 10: Plot of the Horizontal (Total Station Observations) Displacements Magnitudes and their Corresponding Confidence Ellipses

Comparison between the DGPS Observations Displacements Magnitudes and their Corresponding 95% Confidence Ellipses

The horizontal displacements (DGPS observations) magnitudes of the building at three, six, nine, twelve and fifteen months intervals were compared with their corresponding 95% confidence ellipses (Table 2 and Figure 11) to determine if the computed movements between epochs observations were statistically significant. It can be respectively seen from table 2 and figure 11 that the evaluated and plotted displacements magnitudes were all less than their corresponding confidence ellipses implying that the building did not undergo any horizontal displacement during the period of observation vis-à-vis Ehiorobo and Ehigiator (2011).

	MONITORING POINT	K	L	M	N
B/W 1^{ST} & 2^{nd} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000121655	0.000628013	0.000471699	0.000642884
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000866400	0.000781054	0.000989557	0.000766658
	DIFFERENCE (m)	0.000744745	0.000153042	0.000517858	0.000123774
B/W 1^{ST} & 3^{rd} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000756902	0.000403113	0.000700643	0.000516140
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000984888	0.000973116	0.001020895	0.000875003
	DIFFERENCE (m)	0.000227986	0.000570003	0.000320252	0.000358864
B/W 1^{ST} & 4^{th} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000790569	0.000900500	0.00060208	0.000383275
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000875223	0.000912821	0.000948122	0.000647987
	DIFFERENCE (m)	0.000084653	0.000012322	0.000346042	0.000264711
B/W 1^{ST} & 5^{th} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000070711	0.000113137	0.000070711	0.000151327
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.000964392	0.001041570	0.000960400	0.000686280
	DIFFERENCE (m)	0.000893681	0.000928433	0.000889689	0.000534952
B/W 1^{ST} & 6^{th} EPOCHS	MAGNITUDE=SQRT $((\Delta N)^2+(\Delta E)^2)$ (m)	0.000724500	0.000471699	0.000750267	0.000509313
	1.96(SQRT $((SDN)^2+(SDE)^2)$	0.001043228	0.000850965	0.000988781	0.000754027
	DIFFERENCE (m)	0.000318728	0.000379266	0.000238514	0.000244714

Table 2: Comparison of the Horizontal (DGPS Observations) Displacements Magnitudes with their Corresponding Confidence Ellipses

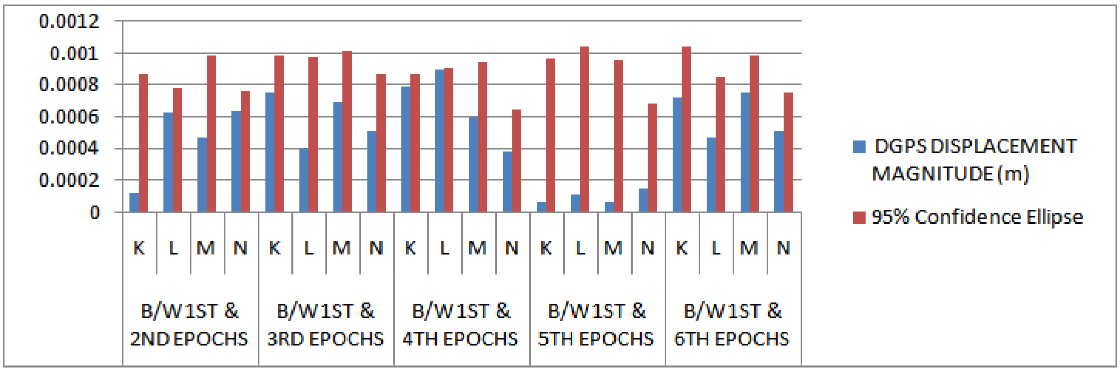

Fig 11: Plot of the Horizontal (DGPS Observations) Displacements Magnitudes and their Corresponding Confidence Ellipses

Comparison of the Two Horizontal (Total Station and DGPS) Methods Accuracy

The two horizontal (total station and DGPS) methods were compared using their a posteriori standard errors (Table 3 and Figure 12) and traces of the variance co-variance matrices (Table 4 and Figure 13) of the six epochs observations to determine which of the two methods is better in terms of accuracy. It can be respectively seen from table 3 and figure 12, and table 4 and figure 13 that the standard errors of unit weight and the traces of the variance co-variance matrices of the six epochs DGPS observations were all less than those of the total station which means that the DGPS method is better than the total station method in terms of accuracy for deformation monitoring of engineering structures.

Table 3: Comparison between the a Posteriori Standard Errors of the *Adjusted Total Station and DGPS Epochs Observations*

A POSTERIORI STANDARD ERROR (m)		
	TOTAL STATION	**DGPS**
EPOCH 1	0.41819	0.002114344
EPOCH 2	0.46066	0.000559521
EPOCH 3	0.45467	0.000463080
EPOCH 4	0.43568	0.002458150
EPOCH 5	0.40733	0.002082528
EPOCH 6	0.47757	0.004780734

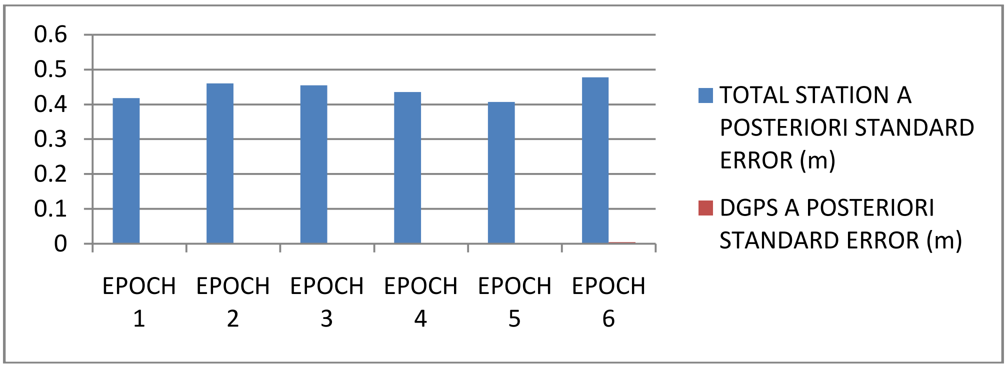

Fig 12: Plot of the a Posteriori Standard Errors of the Adjusted Total Station and DGPS Epochs Observations

Table 4: Comparison between the Variance Co-variance Matrices Traces of the Adjusted Total Station and DGPS Epochs Observations

VARIANCE CO-VARIANC MATRIX TRACE (m)		
	TOTAL STATION	**DGPS**
EPOCH 1	0.000003515	0.000001979
EPOCH 2	0.000002485	0.000000392
EPOCH 3	0.000002420	0.000000223
EPOCH 4	0.000002222	0.000000425
EPOCH 5	0.000001942	0.000000308
EPOCH 6	0.000002670	0.000000510

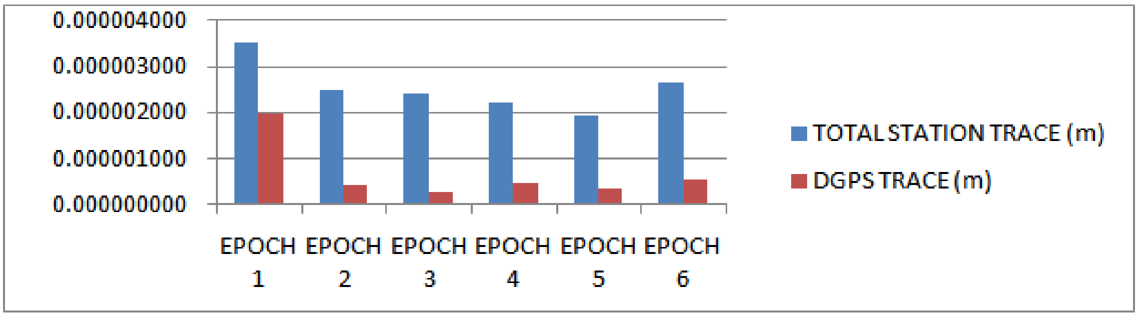

Fig 5.13: Plot of the Variance Co-variance Matrices Traces of the Adjusted Total Station and DGPS Epochs Observations

III Conclusion

Palm House, Benin City was monitored using DGPS and Total Station to determine its structural integrity and to compare the accuracy of the two horizontal methods so as to determine which of them is better in terms of accuracy which in turn will enable users (Surveyors, Geodesist, Engineers, etc.) to decide on the method to employ as regards the purpose of measurement, the magnitudes and direction of the expected displacements of any engineering structure to be monitored. Four reference stations and two sets of monitoring points were used altogether. The monitoring points were observed at six epochs of three months interval using CHC 900 dual frequency GNSS receivers in Static Mode and total station with respect to the reference stations. The DGPS observation were processed using compass software.

The six epochs observations of the two geodetic methods were adjusted with least squares adjustment technique to determine the reliability of the observations and those of the adjusted parameters using Columbus software. The reliability of the observations were determined by carrying out some statistical evaluations. The adjusted observations were accepted as the results of the statistical evaluations and analysis showed that none of the six epochs observations of the two methods was rejected and as the precision and accuracy of each of the adjusted epoch observations and those of the adjusted coordinates were very high.

The adjusted six epochs observations (coordinates) of each of the two methods were compared by finding the differences between the first and the subsequent epochs coordinates. The computed differences in coordinates were used to evaluate the displacements magnitudes of the monitoring points between the first and the subsequent epochs coordinates. The computed displacements magnitudes were compared with their corresponding 95% confidence ellipses to determine if the reported movements were significant or not. The results of the comparison showed that the reported displacements were not significant hence, the building did not undergo any horizontal movement during the period of observation. The two methods were also compared to determine which of them is better in terms of accuracy. The comparison results showed that the DGPS method is better as the a posteriori standard errors and the traces of the variance co-variance matrices of the six epochs DGPS observations were all less than those of the total station.

The results of this study has shown that the building (Palm House) was stable during the period of observation. For this reason, the building is still fit for usage. The study has also shown that the DGPS method is better than the total station method in terms of accuracy for deformation monitoring of engineering structures. This will assist users to decide on the method to apply as the selection of method depends upon the accuracy requirements which in turn are determined by the purpose of measurements and by the magnitude and direction of the expected deformation or movements.

IV Recommendations

Having monitored Palm House, Benin City and compared the accuracy of the two methods, based on the result obtained from this study, the following recommendations were made:

1. That Surveyors, Geodesists, Engineers, etc should employ the most suitable method of monitoring of engineering structures by considering the purpose of the survey and magnitude and direction of the expected deformation (accuracy requirements).
2. That large engineering structures such as high rise buildings, bridges, etc should be monitored at regular basis so as to determine their structural integrity since any movement of the structure which can cause the structure to collapse and thereby result to loss of lives and properties can be detected by epoch monitoring using suitable method and appropriate measures can be taken.
3. That whenever more suitable and accurate method of monitoring of engineering structures is to be employed between the DGPS and the total station methods, the DGPS method should be selected as this study has demonstrated and compared the accuracy of the two methods and showed that the DGPS method is better.
4. That other geodetic methods of monitoring such as InSAR, etc should be compared with any of the two traditional (DGPS and Total Station) methods of monitoring of engineering structures to determine which is better in terms of accuracy.

References

[1]. Abdel-Gawad, A. K., Mogahed, Y. M., Mageed, K. M. A. and El-Maghraby, A. M. (2014): Evaluation and Accuracy Assessment of Static GPS Technique in Monitoring of Horizontal Structural Deformations. *International Journal of Modern Engineering Research*, Vol. 4, No. 6.

[2]. Aghedo, H. O. (2016): Deformation Monitoring of Ikpoba River Bridge in Benin City, Edo State, Using GPS. Unpublished MSc Thesis of the Department of Surveying and Geoinformatics, Nnamdi Azikiwe University,Awka.

[3]. Ameh, B. M. (2013): Determination of Components of Deflection of the Vertical of Lobi Area of Makurdi, Benue State, Using GPS (GNSS) and Precise Levelling Observations. Unpublished M.S.C. Thesis of the Department of Surveying and Geo-informatics, Nnamdi Azikiwe University Awka..

[4]. Aziz, W. A., Othman, Z. and Najib, H. (2001): Monitoring High-Rise Building Deformation using Global Positioning System. Asian GPS Conference.

[5]. Bird, B. (2009): Analysis of Survey Point Displacements Using Total Station Measurements. Published *BSc. Technical Report of the Department of Geomatics Engineering*, British Columbia Institute of Technology.

[6]. Caspary, W. F. (1988): *Concept of Network and Deformation Analysis*. Monogram II, School of Surveying, University of New South Wales, Kensington, Australia. ISBN 0-85839- 044-2.

[7]. Ehiorobo, J. O. and. Ehigiator, R. I. (2011): Monitoring for Horizontal Movement in an Earth Dam Using Differential GPS. *Journal of Emerging Trends in Engineering and Applied Sciences (JETEAS),* Vol., 2 No. 6, 2011, pp 908-913.

[8]. Githumbi, W. J. (2014): Deformation Monitoring of an Oil Storage Tank by Geodetic Methods. Case Study: Oil Libya, Nairobi Terminal. Published B.Sc. Project of the Department of Geospatial and Space Technology, University Of Nairobi.

[9]. Leick, A. (1990): *GPS Satellite Surveying*. John Wiley and Sons.

[10]. Mikhail, E. M. and Gracie, G. (1981): *Analysis and Adjustment of Survey Measurements*. Van Nostrand Reinhold Company, New York.

[11]. Mishima, K. and Endo, K. (2002): The Method of the Design for Survey Network by Q Matrices. Proceedings of the 7th International Workshop on Accelerator Alignment, SPring.

[12]. Ono, M. N., Agbo, J. A., Ijioma, D. I. and Chubado, M. (2014): Establishment of Baseline Data for Monitoring of Deformation of Murtala Mohammed Bridge (MMB) Lokoja Kogi State, Using GPS. *International Journal of Science and Technology,* Vol. 4 No.5, pp 86-92.

[13]. Uzodinma, V. N. (2014): *Deformation Analysis*. Unpublished Lecture Notes of the Department of Surveying and Geo-informatics, Nnamdi Azikiwe University Awka.

Ono, M. N., Eteje, S. O. " Comparative Analysis of Dgps and Total Station Accuracies for Static Deformation Monitoring Of Engineering Structures." IOSR Journal of Environmental Science, Toxicology and Food Technology (IOSR-JESTFT) 12.6 (2018): 19-29.