

Establishment of Deformation and Subsidence Monitoring Baseline in the Coastal Environment: A Case Study of University of Lagos

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Abstract

Deformation and subsidence measurements are very vital for stability of structures and buildings. Deformation and subsidence monitoring are easily carried out with the aid of established baselines. This study focuses on the establishment of baseline for monitoring deformation and subsidence within university of Lagos. Geodetic method of control establishment was adopted, where five (5) control stations were established on stable grounds across the university of Lagos main campus with Differential GPS observation carried out on them and data obtained were processed and analysed statistically. The result of the findings shows that the baseline established is very reliable, given that the vertical controls have their relative redundancy number r_{ij} ranging between $0.1 < r_{ij} < 1.0$ and the standard deviations ranges from 0.002 to 0.005. Also, the relative precision of the established baselines fell within the range of $7.36e-06\text{ppm}$ - $2.54e-05\text{ppm}$. From the findings of this research, deformation and subsidence studies can be reliably monitored within the University of Lagos and its environ using the baseline established through this research in order to safeguard lives and properties – including high rise structures within the university's main campus.

Keywords: *Deformation, Coastal environment, Reference points, Monitoring, Subsidence*

1. Introduction

Ground subsidence can be caused by several geological factors, climatic processes and anthropogenic sources, or by mixture of the above factors. Subsidence is frequently linked to intense faulting and opening of fissures in urban areas, generating a significant geologic hazard that needs to be accurately assessed and monitored (Ferretti et al., 2004; Mazzotti et al., 2009; Brunori et al., 2015). Recent researches have demonstrated the applicability of Global Positioning System (GPS) techniques to precisely determine the 3-D coordinates of moving points in the field of natural hazards such as earthquakes, landslides, and volcanic activity. Indeed, the detailed analysis of the motion of a landslide, in particular for a near real-time warning system, requires the combination of accurate positioning in three dimensions (infracentimetric) and fine temporal resolution (hourly or less) (Malet et al., 2002). Besides, in order to detect and measure the vertical displacement or subsidence of offshore platforms, GPS is considered as the best tool to determine relative position between control

stations because GPS allows us to achieve a desirable precision (i.e. +0.1ppm) that is necessary for subsidence monitoring (Leick et al., 2015)

Techniques of positioning on various time and space scales have made a lot of progress in the last decade, in particular in the field of geomorphological mapping, or in the realization of Digital Elevation Model (DEM) by numerical photogrammetry (Girault, 1992; Miyazawa et al., 2000; Weber and Herrmann, 2000). As a result of the constantly growing technological progress in all fields of engineering, the increasing demand for higher accuracy, efficiency, and sophistication of the deformation measurements, geodetic engineers have continuously searched for better monitoring techniques and have to refine their methods of deformation analysis. The infiltration of space techniques such as GPS has opened a new dimension in data acquisition which involves offshore structures such as gas and oil platforms which are situated hundreds of kilometers offshore (Setan and Othman, 2006). A suitable technique of data acquisition has to be identified such that a high accuracy observation can be obtained and its results can be used for deformation analysis. Amiri-Simkooein et al. (2017) proposed a method that identified the unstable points of a network based on the generalized likelihood ratio (GLR) test. The method simultaneously uses the observations of two epochs called the simultaneous adjustment of two epochs (SATE) method. SATE is applicable to one-, two-, or three-dimensional deformation networks with any type of observations, including distances, angles, global positioning system (GPS) baselines, and height difference. Samsonov et al. (2017) developed a Multidimensional Small Baseline Subset (MSBAS) methodology which is a semi-automatic processing system for computing temporally dense two-dimensional, horizontal east-west and vertical time series of ground deformation from ascending and descending SAR imagery acquired by various satellites. The MSBAS was used for mapping ground deformation at the Piton de la Fournaise volcano (La Réunion Island, France) during the February 2012–April 2016 period from RADARSAT-2 data. The five volcanic eruptions that occurred during the June 2014–October 2015 period, produced over 60cm of horizontal and over 30cm of vertical ground deformation as resolved in the MSBAS-derived time series. The results which were validated with GNSS observations show the high level of accuracy provided by geodetic means of monitoring deformation despite the technological advancements in remote sensing techniques.

A research like this is highly justified in the coastal area like that of University of Lagos because there is need to provide accurate, wide area ground deformation data to complement accurate ground survey data of generally more limited spatial coverage, identify areas of high differential settlement for potential damage to surface infrastructure, provide historic ground movement for baseline and monitoring, identify subsidence depressions that may pose flood risk potential and identify areas of high seismic risk (ground shaking, liquefaction, fault rupture). Coastal environments tend to have weak soil structure resulting from the nature of the vegetation such as mangrove swamps logged with water, hence, the need to monitor structural facilities built in the area using subsidence monitoring baseline (Kirwan and Megonigal, 2013)

This paper focuses on the establishment of a baseline for the purpose of monitoring cases of subsidence and deformation that could likely happen within coastal environment of University of Lagos since population and human activities are increasing geometrically in the area.

2. Literature Review

Over the last decade, interest has grown among structural engineers and other building professionals such as builders, geotechnical engineers, mechanical engineers and surveyors in monitoring the movement of different types of structure both during and after completion of construction. In different parts of the city of Lagos, numerous cases of building collapse have been recorded (Akpan, 2017; Olowopejo, 2018). It is well known that the foundations of large buildings are affected by changes in ground conditions, and also walls of heavy structures change shape with varying pressure (Tasci, 2008). For all these, deformation surveys can be used to measure the amount by which a structure moves both vertically and horizontally over regular time intervals. Although all the principle of many of the techniques used for doing this are recognizable as those used for site surveying and setting out, however, continuous updating of very precise periodic measurements either on structure or a defined location distinguishes a deformation survey from other types of survey (Uren and Price, 1994). Deformation surveys have been carried out to detect or predict subsidence through several techniques of measurement. These methods include Geodetic techniques, Non-Geodetic (Geo-Technical) techniques (Erol et al., 1999), photogrammetric and remote sensing techniques (Rosu et al., 2015).

2.1. Methods of Deformation Monitoring

Geodetic techniques make use of measuring devices that measure geo-referenced displacements or movements in one, two or three dimensions. Geodetic techniques comprise a network of points interconnected by angles and distances measurements. They usually provide a sufficient redundancy of observations, for the statistical evaluation of their quality and for detection of errors (Beshr, 2015). It includes the use of instruments such as total stations, automatic levels, digital levels and global navigation satellite system receivers. The global positioning system is a satellite constellation operated by the United States Department of Defense (Air Force) to support military and civilian positioning, navigation, and timing. The first satellite was launched in 1978 with multiple generations of satellites thereafter. The orbital periods are approximately 12 h, so that the satellites are visible over the same ground point about every 24 h. Currently there are 32 satellites in operation in 6 orbital planes. There are at least five satellites visible to support instantaneous positioning at any geographic location, and often there are up to 12 satellites visible (Bock et al., 2016). Standard civilian applications require the observation of signal propagation time (expressed in terms of equivalent range or distance) from at least four satellites in order to triangulate the 3D position of the user and to estimate a clock/timing bias; additional observations provide redundancy and estimates of position

uncertainties. Other satellite constellations such as the GLONASS by Russia, Galileo by the European Union and BeiDou by China all make up the global navigation satellite system (GNSS) which have similar impact through super redundancy and the number of visible satellites, providing improved positioning in urban and other environments for geodetic, precise surveying, and engineering applications (Langley et al., 2017). Technological advancement in satellite geodesy has led to the recent development of dense and continuously operating Global Navigation Satellite System (GNSS) networks worldwide and as such resulted in a significant increase in geodetic data sets that sometimes capture transient-deformation signals (Walwer et al., 2016)

Several researches have monitored subsidence using the geodetic networks. Kalkan et al. (2015) exploited geodetic techniques to assess possible deformations of the embankment structure of Atatürk dam in Turkey. The geodetic approach included differential, trigonometric, and global positioning system (GPS) leveling observation campaigns over a period of 6.5 years from May 2006 through November 2012. The geodetic control network included 32 reference points and approximately 200 object points located on the surface of the embankment structure. All object points were equipped with forced centering mechanisms to support either optical targets, reflectors, or GPS receivers, depending on the type of geodetic measurement campaign. From the observations, the downstream side of the embankment crest appears more stable, with small vertical movement amplitudes (< 10 cm). Nowel (2015) used the Robust M-estimation in the geometric deformation analysis of geodetic control networks, especially in the analysis of single-point displacements in these networks. Two methods for displacement analysis based on robust M-estimation were used. The first method was the classical robust method, in which the displacement vector is determined from differences in adjusted coordinates. The second method was Generalized Robust Estimation of Deformation from Observation Differences, in which the displacement vector was determined from differences in unadjusted observations. Classical and proposed robust methods were tested on the basis of the simulated two-epoch observations of the absolute control network of the Montsalvens Dam in Switzerland (which is well known in the literature) and on the basis of Monte Carlo simulations. The test results showed that the proposed robust method (Generalized Robust Estimation of Deformation from Observation Differences), in some cases, might be more adequate than the classical robust method, especially when low values of displacements that slightly exceed measurement errors are expected. Scaioni (2018) integrated the data collected with Robotic total stations and GNSS (Global Navigation Satellite System) techniques for measuring 3D displacements on precise locations on the outer surfaces of dams.

Non-Geodetic techniques make use of measuring devices that measure non-georeferenced displacements or movements and related environmental effects or conditions. It includes the use of instruments such as extensometers, piezo-meters, rain gauges, thermometers, barometers, tilt-meters, accelerometers, and seismometers. Non-Geodetic methods have been explored in different researches. Wilczyńska and Ćmielewski (2016) used the accelerometer to detect the direction of the gravity Earth's force field which allows to calculate the angle of inclination from the vertical or

horizontal line between structural elements such as beams. The accuracy of the sensing angle is better than 0.3mrad, which means the ability to determine the deformation better than 0.3 mm/m. They also proposed the use of measuring set comprising laser diode and CCD camera which enables automatic control in real time. The accuracy of the measurement is better than the pixel in the position of the laser beam footprint which was less than 0.3 pixel. Kalkan and Bilgi (2015) adopted the geodetic and non-geodetic methods for monitoring the geometric changes at the Atatürk Dam surface. Physical and geometrical changes in embankment inside were defined using the non-geodetic methods while the bathymetric surveying techniques were also used in the water covered area and Real Time Kinematic (RTK) GNSS surveying technique on the other area in order to determine the topography of the embankment and reservoir surfaces. Marković et al. (2019) comparatively analysed the geodetic and 2D deflection sensing methods for deformation determination. The 2D deflection method is based on fiber-optic curvature sensors (FOCSs). In the geodetic measurements, a geodetic micro-network was established. Measurements were made from a 2D deflection sensor and three total stations for comparison. The analysis of results revealed the potential of 2D deflection sensor application in structural health monitoring (SHM) procedures.

Photogrammetric and Remote Sensing techniques include the use of Earth Observation (EO) satellites such as space-borne optical Very High Resolution (VHR) and Synthetic Aperture Radar (SAR) imageries for displacement measurements using the interferometric principles. Although these spaceborne platforms have revisiting times of but they still cannot match the spatial detail or time resolution obtainable by means of Unmanned Aerial Vehicles (UAV) Digital Photogrammetry (DP), and ground-based devices, such as Ground-Based Interferometric SAR (GB-InSAR), Terrestrial Laser Scanning (TLS) and InfraRed Thermography (IRT) (Casagli et al., 2017). Giri and Kharkovsky (2016) used the non-geodetic method of detecting cracks in concrete which first occur on the surface of concrete structure under load and provide an indication for further degradation. They used a laser displacement sensor (LDS) consisting of LDS mounted on the scanner performing a raster scan. From the observations, the negligible standard deviation proves the repeatability and accuracy of the measurement and shows that the technique can be applied in a real-life scenario. Sun et al. (2015) used remote sensing products for the study of a major landslide in Zhouqu, China which occurred on August 8, 2010. Prior to the landslide, the deformation was studied for about three years by an enhanced StaMPS-SB technique with 16 ascending ALOS/PALSAR images. The deformation in four regions of the study area was retrieved with a maximum of up to 70 mm/yr. Deformation exceeding 30 mm/yr was also detected in the Suoertou slope, which is a well-known place for an ancient fault zone landslide. Schaefer et al. (2016) processed Interferometric Synthetic Aperture Radar (InSAR) images (interferograms) from both spaceborne Advanced Land Observing Satellite-1 (ALOS-1) and aerial Uninhabited Aerial Vehicle Synthetic Aperture Radar (UAVSAR) data acquired between 31 May 2010 and 10 April 2014 to measure post-eruptive deformation events. The detection of several different geophysical signals emphasized the utility of measuring volcanic deformation using remote sensing techniques with broad spatial coverage.

Despite the technological advancement in deformation monitoring techniques, Geodetic techniques have been the most commonly used as a result of improved accuracy of GNSS receivers configured to continuously observe positions known as Continuously Operating Reference Stations (CORS). The Geodetic techniques require accurate computations to the nearest millimeters through adjustment of observations using Least Squares methods.

2.2. Least Squares Adjustments by Observation Equation Method

The functional relationship between adjusted observations and the adjusted parameters is given as (Ono et al., 2014, Ayeni 2010):

$$L^a = F(X^a) \quad [1]$$

Where, L^a = adjusted vector of observations and X^a = adjusted station coordinates. Equation (1) is non-linear function and the general observation equation model is obtained after linearization.

The system of observation equations is presented by matrix notation as (Mishima and Endo 2002):

$$V = AX + L \quad [2]$$

where, A = Design Matrix, X = Vector of Unknowns, L = Calculated Values (l^o) Minus Observed Values (l^b), V = Residual Matrix That is,

$$\begin{matrix} V & & A & & X & + & L \\ \begin{bmatrix} V_1 \\ \dots \\ V_n \end{bmatrix} & = & \begin{bmatrix} a_{11} & a_{12} & \dots & a_{1n} \\ \dots & \dots & \dots & \dots \\ a_{n1} & \dots & \dots & a_{nn} \end{bmatrix} & \begin{bmatrix} x_1 \\ \dots \\ x_n \end{bmatrix} & + & \begin{bmatrix} l_1 \\ \dots \\ l_2 \end{bmatrix} \end{matrix} \quad [3]$$

Applying least squares principles and solve for X, Equation (3) is obtained

$$X = (A^T P A)^{-1} (A^T P L) \quad [4]$$

Where, X = estimate, P = weight matrix

2.3. Least Squares Adjustments by Condition Equation Method

The general case of non-linear model will be treated since the linear model can easily be derived from it. If we define L_a in eqn. (5) as $L_a = L_b + V$ then we have (Ayeni 2010).

$$f(L_b + V) = \hat{O} \quad [5]$$

where L_b , V are Vector of Unadjusted Observations and Vector of Residual respectively.

The Vector of Residual is given as

$$V = P^{-1} B^T K \quad [6]$$

$$V^T P V = -K^T W$$

The formula for a-posteriori variance of unit weight is given by

$$\hat{\sigma}_0^2 = \frac{v^T P v}{r} \quad [7]$$

where r is number of condition equations

$$QLa = P^{-1} - P^{-1} B^T M^{-1} B P^{-1}$$

The Variance-Covariance matrix of adjusted observations is then given as

$$\Sigma La = \hat{\sigma}_0^2 QLa \quad [8]$$

However, this study intends to establish geodetic baselines that would be useful in the deformation monitoring of structural elements of buildings in the University of Lagos coastal habitat due to the vulnerability of the earth formed from sand-filling operations. Other methods of deformation monitoring such as the non-geodetic and remote sensing techniques can be integrated with the geodetic deformation monitoring operation from the established baselines.

3. Study Area

Lagos State is located in the southwestern coast of Nigeria approximately between latitudes 6°22'N and 6°52'N and longitudes 2°42'E and 3°42'E (Odumosu et al., 1999). It is bounded on the west by the Republic of Benin while the southern boundary of the state is formed by the 180km long Atlantic coastline. Its northern and eastern boundaries are shared with Ogun State. The vegetation cover is dominated by swamp forest, wetlands and tropical swamp forest comprising of fresh waters and mangrove. Generally, the pattern of relief in Lagos reflects the coastal location of the state.

The University of Lagos, Akoka, Lagos is located at appropriately latitude 6°30'N to 6°31'N and longitude 3°25'E to 3°27'E in the Lagos Mainland Local Government Area. The University of Lagos, Akoka, is a Federal Government owned University in Nigeria (University of Lagos, 2019). The University of Lagos founded in 1962 is made up of two campuses, the main campus at Akoka, Yaba and the College of Medicine in Idi-Araba, Surulere (University of Lagos, 2019). Both sites are on Lagos Mainland. The main campus which is largely surrounded by the scenic view of the Lagos lagoon and is located on 802 acres (3.25 km²) of land in Akoka, North Eastern part of Yaba, Lagos has been chosen as the study area being a coastal environment which increasing population has led to the increased infrastructural development (Adeniran and Oyelowo, 2013). (Figure 1).



Figure 1. Location of the study area, University of Lagos, Akoka.

4. Materials and Methods

Prior to the execution of the field work, an existing in-situ 1st order control within the campus was located which was used as base station for the field observation. Property Beacons with dimensions 35cm x 35cm x75cm were cast at the five chosen reference points. Each beacon was marked and a prefix number was engraved on it. The project planning phase was essential as the network and deformation monitoring design was initiated. Having multiple control stations in the reference network is critical for improving the reliability of deformation surveys, and for investigating the stability of reference monuments over time. The network design ensured that each control station in the reference network was inter-visible to a maximum number of structural monitoring points (placed on the structure) and to at least two other reference monuments.

Coordinates were transferred by GPS observations from reference station, XSR347 being a 1st order control beacon on campus to the newly established stations (Second Order Controls) in the reference network before the monitoring survey. 3D coordinates were established on all reference stations in the study. The equipment used in the study includes Leica 1200 Differential GNSS (Figure 2a) and Leica DNA 03 Digital level (Figure 2b) for field observation. Methodology employed in this study is divided into three sections: data acquisition, data processing and network adjustment as described in the Methodology flowchart in Figure 3. The existing coordinates of the Reference Station are presented in Table 1.

Table 1. Reference Station Coordinates

Station ID	Easting (m)	Northing (m)	Ortho. Height (m)
XST 347	543235.4300	719894.2200	4.701



Figure 2. (a) Leica 1200 Differential GNSS Set-up (b) Leica DNA 03 Digital level

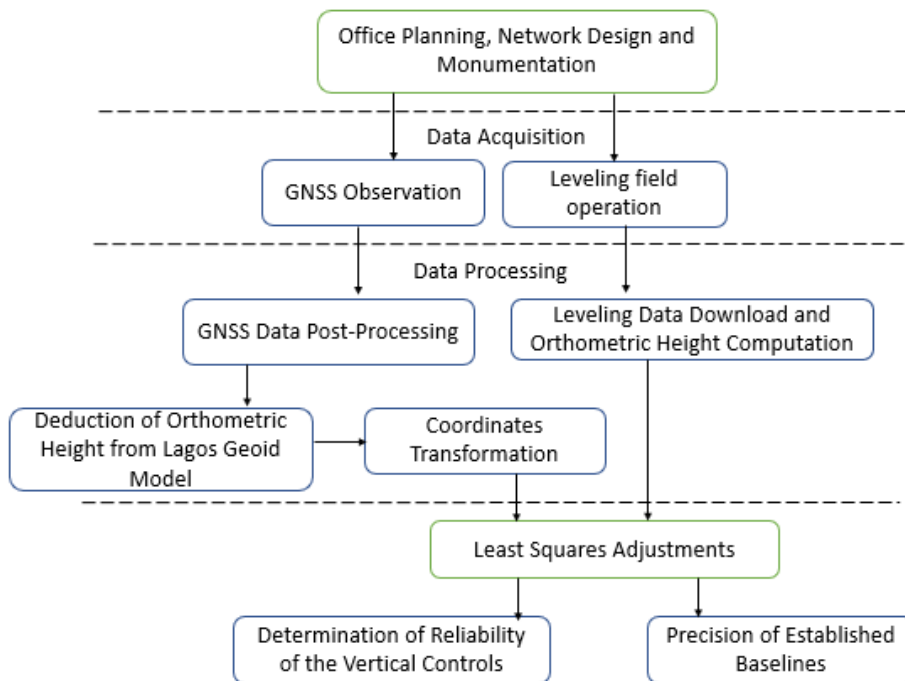


Figure 3. Methodology Workflow

4.1. Data Acquisition

4.1.1. GPS Observation

The rectangular coordinates of the five-point forming the monitoring baseline stations were determined relative to the 1st order control (XST347) with Differential GPS. Observations were taken over a period of two months, the mean of the whole observations on each station were acquired. Digital level instrument was used to carry out levelling from XST347 through the five baseline

stations so as to obtain orthometric height. This first epoch observations of the Differential GPS and digital level were used as reference observations to establish the coordinates (X, Y, Z) of the control points forming the monitoring baseline such that the subsequent epochs observations will be compared with them to ascertain any displacement. The displacement in magnitude and direction of any monitoring structure within the study area could be determined using Equations (9) as the difference in coordinates between the measurement epochs (Ehiorobo and Ehigiator 2011):

$$\begin{cases} X_j^{k+1} - X_i^k = d_x \\ Y_j^{k+1} - Y_i^k = d_y \\ Z_j^{k+1} - Z_i^k = d_z \end{cases} \quad [9]$$

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 monitoring point as:

$$d_s = \sqrt{(d_x)^2 + (d_y)^2}$$

The direction of movement (θ) is computed as:

$$\tan \theta_i = \frac{d_{yi}}{d_{xi}} \quad [10]$$

The method of differential positioning was done using a Leica GPS 1200 + GNSS. The Static mode of operation was adopted with duration of 45minutes. This mode allowed the determination of rover station positions with respect to the Reference station. The base or reference station was set up at XST 347 control point. The data collection procedure or scheme according to USACE (2002) was used at each station as given below;

- a) Session length: A session length of 45 minutes (L1/L2 GPS carrier phase data) was done and this is required to meet minimum positioning accuracies using one observed reference station.
- b) Coverage: A minimum of five (5) visible satellites were tracked at all times. GPS mission planning software was used to maximize the number of continuously tracked satellites in each session.
- c) Station data: Specific information related to the data collection was noted and recorded on the appropriate log sheets at each station.
- d) Recording interval: A five (5) second data logging rate was used in all data collected for monitoring surveys. The logging rate is defined as the time interval (in seconds) between each data value recorded in the receiver's internal memory.

4.1.2. Leveling Operation

The Leica DNA03 digital level was used to transfer height (Orthometric) from a stable reference point (XST 347) to the GPS monument reference points using the principle of differential leveling where the level and the leveling staff are employed to determine vertical distances of points above or below a reference datum. The method of determining elevation using the digital level is through the use of an electronic digital barcode leveling staff to determine the difference in elevation between a known elevation and the height of the instrument, and then the difference in elevation from the height of instrument to an unknown elevation point. The differential leveling operation data were stored in the digital level device due to its ability to retain observation data.

4.2. Data Processing and Adjustments

The GNSS data obtained were post-processed in the Leica Geo Office 4.0 software environment. The Static GNSS observation being a differential observation technique with baselines being "observed" and computed from the reference to the rover was post-processed to provide millimeter-to meter-level precision. Typically, the post-processing involves differential processing relative to a fixed base location. Post-processing the data greatly minimizes or eliminates numerous error sources in GNSS positioning, among which are receiver and satellite clock errors, delay of the GNSS signal through the Earth's atmosphere (most significantly, the ionosphere and the troposphere) (Lauer 2018). The processing parameters were set with the cut-off angle set to 12° from the original 15° for higher accuracy. The coordinate system of the post-processing operation was set to projection UTM 31N based on the WGS84 ellipsoid. The existing coordinates of the base station were keyed in for the baseline processing of the observation set and the post-processed coordinates were generated as the software was set to repeatedly compute the baselines in the network. Processing was done both in the Automatic and Manual modes. In automatic mode, all jobs were selected and processed at once while in the manual processing mode, the rover stations were selected separately from the reference stations and then processed with the intention of ensuring that both sets of observations (Base and Rover) were void of errors or blunders. The Lagos Geoid model developed by Olaleye et al. (2013) was loaded and used to evaluate the orthometric heights of points as ellipsoidal heights are the obtainable from any GPS observation.

The downloaded levelling data from the Digital level were imported into MatLab environment for adjustment based on the principle of least squares models presented in Equations (6) to (8). Table 2 shows the result of the levelling network which are the distances between the stations as well as the height difference between two stations.

Table 2. Leveling baseline distance and Change in Height

Station	Distance (m)	dH (m)
XST347-GME02	800.752	2.5420
GME02-GME03	202.456	0.2048
GME03-GME04	182.013	0.7154
GME04-GME05	144.098	-0.2014
GME05-GME06	114.920	-1.0645
GME06-GME03	369.739	0.5414
GME03-GME02	202.456	-0.1975
GME02-XST347	800.752	-2.5485

Where dH is the height difference between stated points and was obtained from observations made on site and Distance is the linear distance between stated points as obtained from coordinates of points ($\sqrt{\Delta E^2 + \Delta N^2}$). The height of known control, XST347 is given as 4.701 meters.

4.3. Coordinate Transformation

Coordinates were converted from UTM WGS84 datum to UTM Nigerian Local Datum (Minna Datum) using GeoCalc (Geographic Calculator) software. The GeoCalc software is a coordinate transformation software readily available for coordinate transformation and has been used in different researches (Kumar and Murry, 2017; Eteje et al., 2018). The Coordinates were transformed to the UTM Nigerian Local Datum (Minna Datum) to ensure coherence with the locally used coordinate system as the baselines will be used for monitoring engineering structures in the local space.

5. Results and Analysis

This section presents and discusses the results from the observations and post-processes. Table 3 shows the results of the processed coordinates using Leica Geo Office 4.0 software along with the standard deviations in Easting and Northing.

Table 3. 3D Coordinates of Points and Standard Deviation

Station ID	Easting (m)	Northing (m)	Ellipsoid Height, h(m)	Std. Dev. Easting (m)	Std. Dev. Northing (m)
GME02	543971.8901	720208.5936	30.0777	0.0023	0.0023
GME03	543938.7781	720408.3233	30.2855	0.0017	0.0016
GME04	543885.2924	720582.2999	31.0157	0.0021	0.0019
GME05	544022.5950	720626.0305	30.8074	0.0028	0.0026
GME06	544110.0961	720551.5312	29.7357	0.0023	0.0024
XST347 (Reference)	543235.4300	719894.2200	27.5045	0.0008	0.0007

Table 3 reveals the consistency of the repeated observations on each of the observation stations as standard deviation. The repeated observations were based on the data logging interval. The standard deviation shows that the base station has the most consistent set of observations due to the station occupation time. The standard deviation reduces with increased number of observations. Among the rover stations, GME 03 has the highest consistency of standard deviation values of 0.0017 and 0.0016 in easting and northing respectively while GME 05 is least consistent. This standard deviation value is not unconnected with the baseline accuracy which could be as a result of several factors such as baseline distance, observation setup stability, atmospheric conditions, multipath sources.

Note, however that the orthometric heights in Table 2 were obtained using the Lagos Geoid model. The orthometric heights obtained from actual leveling operations were adjusted with least squares according to Ayeni (2010). Table 3 shows the height differences between stations.

Table 4 shows the differences between the Geoid derived orthometric heights and the adjusted heights obtained from actual leveling operation.

Table 4. Difference in the Orthometric Heights from Field Observations and Lagos Geoid Model

Station ID	Orthometric Height, H(m) (Field Observation)	Orthometric Height, H(m) (Lagos Geoid)	Difference in Height (m)
GME02	7.2553	7.2462	0.0090
GME03	7.4564	7.4474	0.0090
GME04	8.1738	8.1648	0.0090
GME05	7.9741	7.9651	0.0090
GME06	6.9109	6.9018	0.0090
XST347 (Reference)	4.7010	4.7010	0.0000

Table 4 shows the equal difference in the height referenced to the Geoidal model obtained from the GNSS post-processed result (adjusted) and the orthometric heights (adjusted) obtained from the leveling operation. The difference between the mean sea level and the derived Lagos Geoidal model is seen to be 0.0090metres. Trends in deformation detection can be observed as consistent observations taken using the baseline monuments established in this research. The significance of these initial results is enhanced when considering the reliability of the established baselines judging from their accuracies and precisions acquired with the research.

5.1. Reliability of Vertical Controls

Reliability refers to the controllability of observations i.e. the ability to detect blunders in the observations (Kurotamuno, 2016). According to Ayeni (2010), the amount of redundancy (r_{ij}) that each observation adds to the solution is given by:

$$r_{ij} = q_{ij} p_{ij} \tag{11}$$

Where r_{ij} is the observational redundancy number.

q_{ij} is the i th diagonal element of the observational weight matrix P

p_{ij} is the i th diagonal element of the covariance matrix of the residuals

The matrices of q_{ij} and p_{ij} have been computed in the previous section, therefore, the computation of the amount of redundancy gives the result below;

$r_{ij} =$

$$\begin{pmatrix} 0.5000 & 0 & 0 & 3.9448e-16 & 4.9464e-16 & -3.0748e-16 & 0 & 0.5000 \\ 0 & 0.5000 & 0 & 0 & 0 & 0 & 0.5000 & 0 \\ 0 & 0 & 0.2245 & 0.2245 & 0.2245 & 0.2245 & 0 & 0 \\ -7.0988e-17 & 0 & 0.1777 & 0.1777 & 0.1777 & 0.1777 & 0 & -7.0988e-17 \\ -7.0988e-17 & 0 & 0.1417 & 0.1417 & 0.1417 & 0.1417 & 0 & -7.0988e-17 \\ -1.4198e-16 & 0 & 0.4560 & 0.4560 & 0.4560 & 0.4560 & 0 & -1.4198e-16 \\ 0 & 0.5000 & 0 & 0 & 0 & 0 & 0.5000 & 0 \\ 0.5000 & 0 & 0 & 3.9448e-16 & 4.9464e-16 & -3.0748e-16 & 0 & 0.5000 \end{pmatrix}$$

From least squares estimate the highest redundancy number was 0.5 which is the height difference between XST347 and GME02 and the lowest redundancy number is 0.1417 which is the height difference between GME05 and GME06. From the results, it can be concluded that the established vertical controls are highly reliable.

5.2. Precision of Established Baselines

In computing the relative accuracy of each of the newly established baselines, the relative precision of a traverse leg formula was adopted from Ghilani and Wolf (2012) and modified to yield the formula below (Table 5).

$$\text{Precision} = \sigma_A / L_A \tag{12}$$

Where:

σ_A is the standard deviation of the baseline between controls in consideration

L_A is the length of baseline between controls in consideration.

Table 5. Precision of Established Baselines

S/N	Baseline	Baseline length (m)	Precision in Part Per Million (PPM)
1.	XST347 to GME02	800.752	7.36e-06
2.	GME02 to GME03	202.456	1.46e-06
3.	GME03 to GME04	182.013	1.92e-05
4.	GME04 to GME05	144.098	2.22e-05
5.	GME05 to GME06	114.920	2.54e-05
6.	GME06 to GME03	369.739	1.13e-05

Part of the results of establishment of baseline for monitoring deformation in University of Lagos as detailed in Table 5 show that the best precision was acquired between GME02 to GME03 which is $1.46e-05$. The results are in correlation with the standard given in the work of Malet et al. (2002) and Gili et al (2000) where surficial displacements and their precision using GPS is found to be between 1-2mm for a typical baseline range of less than 20km. Furthermore, inferring from the accuracy claimed by different deformation monitoring techniques in the work of Savvaiddi (2003), GPS L1/L2 static observation must give a typical accuracy of $\pm (5\text{mm} \pm 2\text{ppm})$ for a distance of less than 50km between two stations and $\pm (1-3\text{mm} \pm 2\text{ppm})$ for a distance of less than 1 – 2km between two points. Comparing the results in Table 3 from what is the standard from the literature, the results obtained from this research show that the baseline is standard and very reliable to carry out deformation monitoring not in the University of Lagos alone but around its environ.

6. Conclusions

Deformation and subsidence measurements are very vital for stability of structures and buildings. Deformation and subsidence monitoring are easily carried out with the aid of well-established baselines. This study focuses on the establishment of baseline for monitoring deformation and subsidence within university of Lagos. Geodetic method of control establishment was used, where five (5) control stations were established on stable grounds across the university's main campus with Differential GPS observation carried out on them and data obtained were processed and analysis statistically. The result of the findings shows that the baseline established is very reliable given that the vertical controls have their relative redundancy number r_{ij} ranging between $0.1 < r_{ij} < 1.0$, and the standard deviations ranges from 0.002 to 0.005. Also, the reliability of the established baselines fell within the range of $7.36e-06\text{ppm}$ - $2.54e-05\text{ppm}$. From the findings of this research, deformation and subsidence monitoring studies can be carried in University of Lagos corridor in order to safeguard lives and properties in the environment – including high rise structures within the university's main campus.

With the economic and social importance of the coastal environment to the growth and well-being of the inhabitants, which includes implementation of marine and coastal structures and activities, there should be adequate and regular monitoring measurement to forestall any occurrence of deformation on the heavy structures around and within the University of Lagos.

7. References

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