The Use of Geodetic Leveling For Crustal Motion and Deformation Studies: A 30-Year Case Study in Ahmadu Bello University, Zaria

By

O. A. Isioye* and A. A. Musa**

*Department of Surveying and Geomatics Engineering
Faculty of Engineering, Ahmadu Bello University, Zaria, Nigeria

**Currently at: Department of Information and Finance
Lancashire Business School, University of Central Lancashire, Preston PR1 2HE, United Kingdom

ABSTRACT

This paper examines the stability of some benchmarks on the main campus of Ahmadu Bello University, Zaria, and reports the deformation studies of the area around the University dam. This dam serves as the main storage reservoir for the water supply to the University and surrounding communities. In order to study the deformation around the University dam and the stability of benchmarks, several surveys were carried out between 1976 and 2006. Within the period under review, it was revealed that the maximum downward displacement (settlement) of 96.44mm was recorded between 1976 and 2006 for a monitoring station (CP4) situated centrally along the embankment of the dam. The maximum upward displacement (uplift) of 8.03mm was recorded between 1976 and 2002 at a ground point (CP12) located in a roundabout in front of Faculty of Medicine of the University. The monitoring Station CP4 had the greatest average absolute displacement of 3.22mm per annum over the period of thirty years. Detailed results and analysis will be discussed in this paper.

Introduction

The earth's terrain is not at rest but moves slowly due probably to the nature of the earth itself and the underground man-made conditions of the earth's crust. It is, therefore, very important to be sure of the movements of engineering structures that serve human life. Hence, a lot of deformation monitoring studies for determining and analyzing different kinds of engineering structures such as high-rise buildings or infrastructures, dams, bridges, viaducts, industrial complexes, etc. are often implemented. During these studies, the used measurement techniques and systems, which could be geodetic or non-geodetic, are determined considering the type of structure to be monitored, its environmental conditions, and the expected accuracy from the measurements. A few causes of such movements are volcanic eruption which moves downwards on a sloping rock formation, ultimately causing a landslide; change of ground water level, which may result in compression (compaction or consolidation) of the interceded layers of clay and silt the aquifer system; drainage and oxidation of organization soils; dissolution and collapse of susceptible rocks; tidal phenomena; tectonic phenomena; etc. Some serious effects of crustal deformation include loss of level free board and subsequent reduction in flood protection, change in gradient along water conveyance canals, and collapse of engineering structures (Richardus, 1977).

Monitoring and analyzing deformations of engineering structures (such as dams, bridges, viaducts, high-rise buildings, etc.) constitutes a special task for geodesists. There are several techniques for measuring the deformations. These techniques and instrumentation for deformation measurement have traditionally been categorized into two groups: (a) geodetic surveys, which include conventional (terrestrial, such as precise levelling measurements, angle and distance measurements, etc.), photogrammetric (terrestrial, aerial and digital photogrammetry), satellite (such as GPS, InSAR) and some special techniques; (b) geotechnical/structural measurements, using lasers, tilt meters, strain meters, extensometers, joint-meters, plumb lines, micrometers, etc. Each main measurement technique has its own advantages and drawbacks. Geodetic techniques, through a network of points interconnected by angle and/or distance measurements, usually supply a sufficient redundancy of observations for the statistical evaluation of their quality and for detection of errors. They give global information on the behaviour of the deformable structure, while the non-geodetic techniques give localized and locally disturbed information without any check unless compared with some other independent measurements. On the other hand, the instruments, which are used in non-geodetic measurements, are easier to adapt for automatic and continuous monitoring than conventional instruments of geodetic measurements. Geodetic techniques have traditionally been used mainly for determining the absolute displacements of selected points on the surface of the object with respect to some reference points that are assumed to be stable. Non-geodetic techniques, on the other hand, have mainly been used for relative deformation measurements within the deformable object and its surroundings (US Army Corps of Engineers, 2002).

Over the last decade, surface deformation studies at subduction zones have focused primarily on GPS-determined horizontal deformation. The emphasis on horizontal deformation is because GPS-determined vertical deformation rates do not have sufficient precision to constrain subduction models (Verdonck, 2004). This is because the height component is the least accurately determined GPS coordinate, predominantly due to inherent geometric weakness and atmospheric errors (Featherstone et al, 1998; Celik et al., 2001). From the foregoing, precise levelling (or geodetic levelling) can be seen as an indispensable tool in deformation monitoring even in the present days of GPS.

In this study, vertical deformation analysis of benchmarks within the main campus of Ahmadu Bello University, Zaria, using precise levelling measurements is implemented using results that cover a period of 30 years. A review of geodetic techniques of deformation analysis is given in addition to explanation on used adjustments method, analysis methods and results. Results of the study indicated that over 75% of the monitoring stations tested in 2006 were displaced downward, with the greatest movement of 96.44mm between 1976 and 2006 recorded at a station named CP4, located centrally along the embankment of the dam. Benchmarks located upland are covered by another network, where about 70% of the benchmarks were displaced upward (uplifted) between 1976 and 2002 with a maximum value of 8.03mm between 1976 and 2002 at a ground station name CP13. These results when compared to those obtained by Ebong and Musa (1992) show that more movements have occurred over the last 14 years. The causes of these movements are investigated in this paper.

Overview of Geodetic Methods of Deformation Measurement

Deformation monitoring is conducted for the purpose of detecting and interpreting small changes in the geometric status of the earth. With the rapid developments in the field of modern geodesy and with the unprecedented accuracy achievable in geodetic measurements using advanced techniques, the geodetic methods have gained world-wide acceptance for monitoring crustal dynamics. The following sections give short descriptions of the geodetic methods of deformation measurements.

Photogrammetry

If an object is photographed from two or more survey points of known relative positions (known coordinates) with a known relative orientation of the camera(s), relative positions of any identifiable object points can be determined from the geometrical relationship between the intersecting optical rays, which connect the image and object points. Aerial photogrammetry has been extensively used in determining ground movements in ground subsidence studies in mining areas, and terrestrial photogrammetry has been used in monitoring of engineering structures. The main advantages of using photogrammetry are the reduced time of field work, simultaneous three-dimensional coordinates, and in principle an unlimited number of points can be monitored (US Army Corps of Engineers, 2002).

GPS

The Global Positioning System (GPS) has been used extensively and with great success for the production and propagation of survey control. During the development of surveying by GPS the focus was typically on horizontal control with the ability of GPS to measure height seen as an added extra. GPS surveying has now matured to the point where it is seen as a true three-dimensional tool. However, application of GPS to the measurement of height can be complex and solving the problems involved can account for the majority of the effort in finalizing a GPS surveying project (Higgins, 1999).

GPS measures heights related to the ellipsoid. In some cases, ellipsoidal heights alone are sufficient for the type of survey being undertaken. However, many applications require heights that are related to a physically meaningful surface such as the geoid or at least some attempt at realizing the geoid such as a surface based on locally observed mean sea level. Such physically meaningful heights take the form of orthometric or normal heights. While many applications for GPS surveying need to produce orthometric or normal heights, there are some applications where ellipsoidal heights alone are useful. One such application is vertical deformation monitoring where the most important issue is to quantify a change in height over time and whether any change is relative to the geoid or ellipsoid is not particularly relevant (Higgins, 1999), in so far as the factors responsible for the change in height are not being sought.

Using GPS for deformation monitoring brings the normal advances of GPS surveying, such as no requirement for intervisibility between stations and the ability to span large distances with high precision. Also, deformation applications require many repeated observations over time and GPS is well suited to automated survey processes that can significantly reduce cost. Issues to be considered in designing a GPS deformation survey include ambiguity resolution, quality of ephemeris and starting coordinates, multi-path, troposphere, tidal phenomena, antenna phase centre modeling, and antenna height measurement.

InSAR

Elevations can be determined from Synthetic Aperture Radar (SAR) images by interferometric methods. Interferometric Synthetic Aperture Radar (InSAR) is an increasingly popular alternative to GPS or conventional surveying methods. InSAR is a space-borne remote sensing technique that uses changes in satellite radar signals created by interferences on the earth's surface to measure changes in land surface elevation. It is used to measure and track deformations in the earth's surface caused by earthquakes, volcanoes, and by groundwater and fossil fuel extraction and injection. Similar to GPS, InSAR enables measurement of subsidence on a regional scale; the accuracy of elevation measurements with InSAR can be within 2 cm. InSAR is today a cost effective means of monitoring subsidence (Gabriel et al., 1989).

DInSAR

The most accurate form of interferometric measurement is differential interferometry (DInSAR). The DInSAR techniques exploit the information contained in the radar phase of at least two complex SAR images acquired in different epochs over the same area and that form an interferometric pair. Unlike a simple amplitude SAR image, which only contains the amplitude of the SAR signal, a complex SAR image contains two components per pixel, from which the amplitude and phase signal can be derived. The phase is the key observable of all interferometric SAR techniques. The repeated acquisition of images over a given area is usually performed by using the same sensor, e.g., the Envisat ASAR or sensors with identical system characteristics, as it is the case with ERS-1 and ERS-2. Only in particular cases, is it possible to make cross interferometry by using images acquired with different systems (Amaud et al, 2003). It is claimed that height differences as small as one centimeter can be detected by this method. Such a technique, therefore, has the potential of being a cost effective, near-continuous, remote method of measuring terrain subsidence due to mining, and ground movements due to land subsidence, earthquake or volcanic activity.

Laser Scanning

Surveying, including GPS, methods used to monitor large structures such as buildings or infrastructures, while very accurate, are greatly hindered by their low point density. For such traditional techniques, data acquisition time limits monitoring to only a few samples located at strategic points on the structure. Ground-based laser scanning is a new technology that allows rapid, remote measurement of millions of points, thus providing an unprecedented amount of spatial information. This in turn permits more accurate prediction of the forces acting on a structure. But as an emerging technology, several issues concerning instrument calibration, sensitivity analysis, data processing and data filtering techniques require investigation (Amand et al, 2003).

Precise Spirit Levelling

The operation of precise spirit levelling is directed to the determination of the elevation of a number of vertical control points from which those of other points throughout the survey may be determined. The elevations required are absolute elevations from the datum of mean sea level (MSL), and unless the levelling can be connected to that of an adjoining system, it will be necessary to establish the datum by observation. However, in Nigeria the datum had been determined as far back as 1954 from the only tidal station then at Lagos. There is no difference in principle between precise and ordinary levelling; in the latter the distances run between checks are relatively short and with the usual precautions the results are sufficiently accurate for everyday purposes. For precise levelling, on the other hand, the circuits may be conducted so that the uncertainty of each individual determination as well as the actual closing error is reduced to a minimum. It is important to note that success in precise levelling depends upon a due appreciation of the nature and relative importance of systematic errors and their treatment (Clarke, 1978). The instruments required for geodetic leveling do not in principle differ from those for ordinary levelling, barring some special manufacturing attention paid to some constructional details to improve their precision.

Generally, geodetic levelling can be classified in terms of height misclosures as first (primary) order, second (secondary) order and third (tertiary or ordinary) order. The accuracy obtained depends largely on the techniques of observation, instrument, nature of the terrain, and requirement or purpose of the work. The accuracy standards and general allowable misclosures between the forward and backward geodetic levelling runs are shown in Table 1 below.

Table 1: National Vertical Control Accuracy Standards (SURCON, 2003)

Classification	***First Order***	***Second Order***	***Third Order***
Accuracy Standards	$0.6mm/\sqrt{km}$	$1.1mm/\sqrt{km}$	$3.0mm/\sqrt{km}$
Principal Uses	Establishment and development of the main vertical control network of Nigeria, extensive engineering projects, and scientific investigation	Breakdown of first order vertical control network, large scale engineering projects, extension of national networks to un - surveyed areas, and scientific studies	Breakdown of second order, miscellaneous ordinary engineering projects
Method of Realization	Geodetic levelling and gravity measurements	Geodetic levelling	Geodetic levelling, vertical angle measurements, and GPS observations
Maximum Closure for Sections(Forward and Backward)	$2.8mm/\sqrt{km}$	$8.4mm/\sqrt{km}$	$24mm/\sqrt{km}$
Maximum Closure for Loops	$4mm/\sqrt{km}$	$8.4mm/\sqrt{km}$	$24mm/\sqrt{km}$

First order levelling is required in the establishment of a height system over a large area and in deformation monitoring of infrastructures. In both cases, the spherical surface of the earth is taken into consideration and so is the effect of coefficient of refraction. This was the method used for the projects being reported in this paper.

Description of the Networks And Data Collected

In this study, the deformation around the University dam and the stability of benchmarks around the University main campus were investigated using precise levelling. The levelling for each epoch of observation was carried out systematically, after sound preparations, exclusively with the Wild N3 level and invar centimeter staffs with double scale.

Henceforth, in this study the network around the University dam shall be tagged network A, while benchmarks upland form network B. It is important to note that the levelling networks reported in this paper do not include those already reported in Ebong and Musa (1992). It was reported in the latter paper that the first levelling network for the study of deformation on the University campus was established in 1976. This network, which initially consisted of 17 lines and 10 junctions points with a total length of 11km, was extended in 1979 by the addition of new benchmarks, bringing the total length of levelling to 18km. repeated levelling were carried out in 1980, 1983 and 1990.

The Ahmadu Bello University dam was built in 1973 across Kubanni River in Zaria. The dam is 839.42m long. The depth of the dam by its wall is 6.59m, while its height from foundation level is 10.60m. The dam is an earth/gravity dam. The water level at the peak of the rainy season (August) is usually about

4.70m, while it is 2.70m during dry season (March). The reservoir covers an area of 80.94 hectares and has a storage capacity of 2.4 million cubic meters (Ologe, 1973).

The deformation measurements of network A involved four measurement campaigns (epochs of observations). The first campaign was carried out in 1976, the second in 1986, the third in 1992, and the last one in 2006. These four campaigns were aimed at investigating the deformations of the dam and its surroundings. Before carrying out any measurement campaign, a well-designed levelling network was established, the level instrument tested in 2- and 3-peg tests, and the tapes standardized. The characteristics of the soil in the area of investigation were also tested in 1992 for both networks. However, results of the soil tests are not reported in this paper; they can be found in Sule (1992) and Olatunji (1992). A summary of the geodetic levelling data/statistics for network A is given in Table 2(a) below.

Table 2 (a): Statistics for Network A

Year	*No. of Monitoring Stations*	*No. of Loops*	*No. of Lines*	*Total Distance (km)*
1976	8	7	14	9.98
1986	8	7	22	9.51
1992	11	16	26	10.39
2006	25	23	47	21.11

Network B involved five measurement campaigns. The design of network B is very similar to that of network A. The major difference between the two networks lies in their locations. Table 2 (b) summarizes the geodetic levelling data/statistics for network B.

Table 2 (b): Statistics for Network B

Year	*No. of Monitoring Stations*	*No. of Loops*	*No. of Lines*	*Total Distance (km)*
1976	11	16	17	11
1991	5	4	7	6.38
1992	9	13	21	9.7
1993	7	10	16	7.9
2002	9	10	16	8.16

Figures 1(a) and (b) below show the design of the levelling networks for the last epochs of observations for networks A and B, respectively.

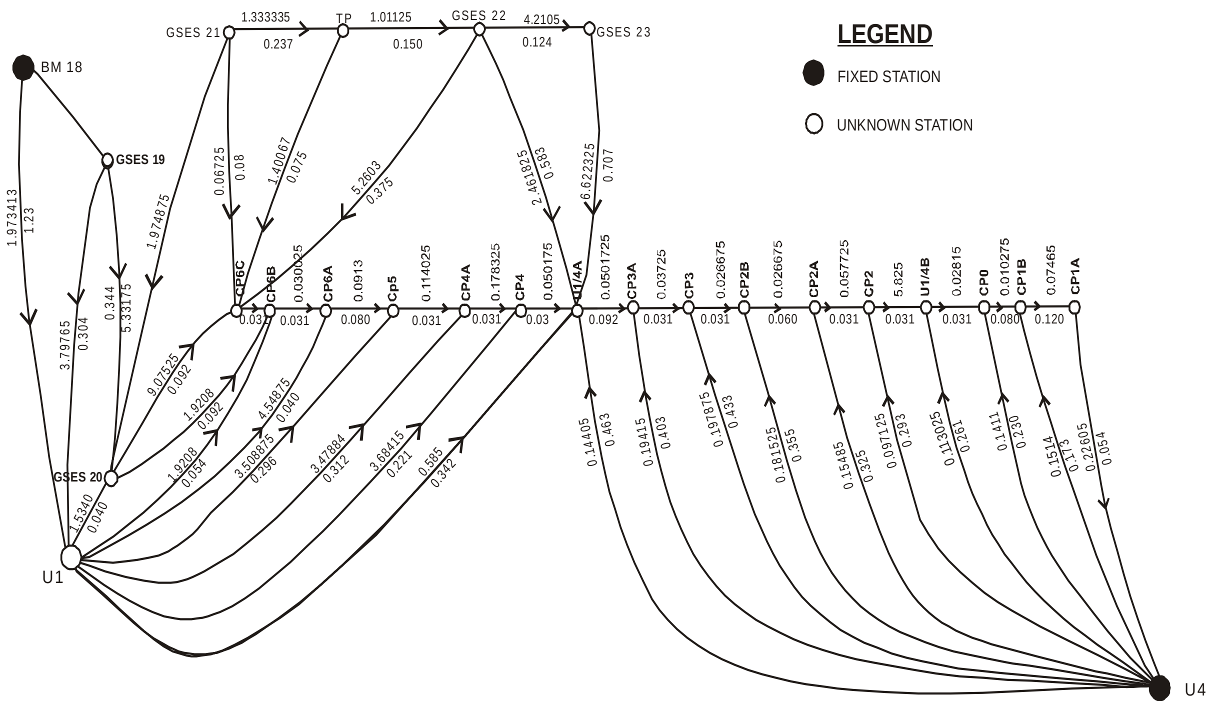

Figure 1(a): Observed Height Differences and Distance between benchmarks for 2006 survey (Network A) (Not to Scale)

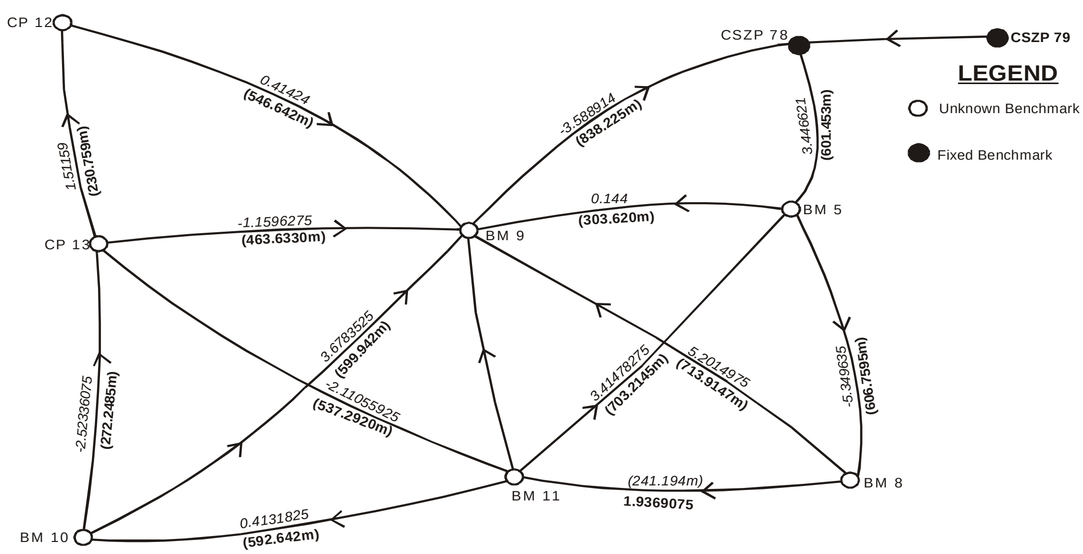

Figure 1(b): Observed Height Differences and Distance between benchmarks for 2002 survey (Network B) (Not to Scale)

.

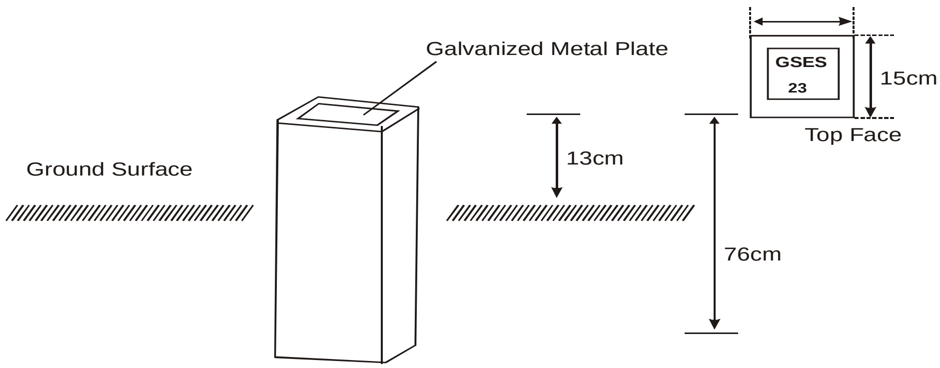

Figure 2: New Benchmark (GSES-Series) introduced in 2006

The type of benchmarks and monitoring stations used in this paper are the same as those already reported by Ebong and Musa (1992), except those of the 2006 survey in which a new type of benchmark (GSES series) was introduced; The GSES series of benchmarks are pre-cast concrete pillars measuring 15cm x 15cm in section and 76cm long. They consist of a 5mm diameter iron rod of length 74cm, with a galvanized metal plate measuring 10cm x 10cm embedded on the top surface of the pillars. The pillars were buried underground to a depth of 63cm in cement mortar and 13cm above the ground level. The concrete mix of 1:2:3 recommended by SURCON (2003) was used for the construction of these benchmarks. Figure 2 depicts this type of benchmark (Anosike, 2007)

Adjustment of Levelling Networks

The approximate heights of monitoring stations/benchmarks obtained in the previous section were adjusted using least squares adjustment method coded in FORTRAN computer language. The program was originally developed by Musa (1992), where the details of the models used can be found.

The accuracy of each observation campaign for both Networks A and B was deduced from the mean accidental error, using (Ebong 1981)

$$M_1^2 = \frac{1}{4} N_r \sum \frac{\Delta^2}{r}$$

where Δ is the level discrepancy in mm between two benchmarks separated by a distance r in kilometres and N_r is the number of discrepancies involved in each level network.

The accuracy of each levelling network surpassed the minimum stipulated value of $0.6mm/\sqrt{km}$ given by the SURCON (2003), and thus they met the geodetic requirements for first order levelling in Nigeria.

Deformation Analysis Using Height Differences

In Ebong and Musa (1992), the height differences of five benchmarks (BM3, BM9, BM10, BM11, and BM19) between 1976 and 1983 were analysed. It was revealed that the maximum and minimum movements recorded during this interval of time were 0.67mm and 0.08mm. Thus, in no case was the movement up to 1mm. The significance of this is that the benchmarks were in fairly stable location. The vertical movements that occurred at these benchmarks in the 7-year period are given in Table 3. With regard to the four test benchmarks (BMRJ1, BMRJ2, BMRJ3, and BMRJ4) which were levelled in 1979, 1980, and 1990, the height differences contained in Table 4 show that these marks were continuously moving during their first year. The 1983 levelling over the test benchmarks shows that the benchmarks had reached their levels of stability in 1983. In 1990 only BMRJ1 and BMRJ3 could be found; the apparent movements indicated at these marks were uplifts of 0.39mm and 0.87mm at the two marks, respectively. No great value was attached to these results because of the inherent weakness in the structure of the 1990 levelling loop from which the apparent movements were deduced.

Table 3: Vertical Movements of Benchmarks between 1976 and 1983(after Ebong and Musa, 1992)

Benchmarks	*BM3*	*BM9*	*BM10*	*BM11*	*BM19*
1976-1983 (mm)	− 0.20	− 0.08	+ 0.64	+ 0.27	+ 0.67

[(+) indicates upliftment and (−) settlement]

Table 4: Vertical Movements of the Test Benchmarks between the Base Year (1979) and Other Epochs (after Ebong and Musa, 1992)

Test Benchmarks	*1979-1980 (mm)*	*1979-1983 (mm)*	*1979-1990 (mm)*
BMRJ1	− 3.30	− 0.53	+ 0.39
BMRJ2	− 1.38	− 0.40	▪
BMRJ3	− 1.19	− 0.25	+ 0.87
BMRJ4	− 2.79	▪	▪

It is important to note that observational procedures adopted for the surveys reported in this paper are as described in Ebong and Musa (1992). The first complete measurement is usually referred to as the reference measurement. Considerable changes in the definition of constants and unknown parameters, as well as greater modifications of the stochastic model, may have unwanted consequences on deformation results. It was for these reasons that the same observational and adjustment techniques were adopted for all levelling campaigns in this paper.

In this study, heights of monitoring stations/benchmarks were compared to their reference values. Tables 5(a) and (b) below show the results of the comparisons for networks A and B, respectively. The results have been obtained using the data of Isa (1978), Oluyori (1987), Olatunji (1992), and Anosike (2007) for network A; and Arinola (1976), Soile (1979), Aladewolu (1991), Sule (1992), Ojigi (1993), and Onuche (2002) for network B. The analysis of the results for networks A and B are contained in sections 5.1 and 5.2, respectively. Note that the results presented below exclude those already discussed in Ebong and Musa (1992), whose summary is already contained in Tables 3 and 4 above.

Table 5(a): Displacements Between Reference Measurement (1976) and Other Epochs (Network A)

Monitoring Stations	*1976-1986 (mm)*	*1976-1992 (mm)*	*1976-2006 (mm)*
CP1	− 1.27	▪	− 8.48
CP2	− 0.69	− 5.39	− 5.29
CP3	− 2.00	− 3.02	− 1.53
CP4	− 1.21	− 64.35	−96.44
CP5	▪	− 1.49	+ 0.21
U1	▪	− 0.53	+ 0.22
U4	▪	− 0.01	− 5.76
BMA	− 1.62	− 0.43	▪
BMB	− 3.29	▪	− 5.30

Table 5(b): Displacements between Reference Measurement (1976) and Other Epochs (Network B)

Benchmarks	*1976-1991 (mm)*	*1976-1992 (mm)*	*1976-1993 (mm)*	*1976-2006 (mm)*
BM5	− 14.20	− 15.29	− 11.77	− 5.58
BM9	+ 5.87	+ 1.47	+ 1.69	+ 6.35
BM10	▪	▪	▪	+ 3.66
BM11	▪	+ 4.86	▪	+ 6.90
BM18	▪	▪	▪	− 10.00
CP12	− 4.32	− 9.06	− 4.69	− 18.91
CP13	▪	▪	+ 2.40	+ 8.03

NETWORK A

1976-1986 LEVELLING

The movements of six monitoring stations, which were observed in the levelling of 1976 and August 1986 and the vertical movement that occurred at these stations in the 10-year period are contained in Table 5(a). These results show that there were settlements at all the monitoring stations. The maximum settlement of 3.29mm was recorded at BMB and minimum or smallest settlement of 0.69mm was recorded at CP2. The little movements can be attributed to the weight of the benchmarks, which must have taken place few years after emplacement and consolidation of the soil particles. The significance of these movements is that the monitoring stations were in stable conditions between 1976 and 1986.

1976 – 1992 LEVELLING

The movements of seven monitoring stations that were observed in the levelling of 1976 and 1992 and the vertical movements that occurred at these stations in the16-year interval are contained in column 3 of Table 5(a). The results are quite similar to those obtained in 1986. There was settlement at all the stations. CP4 had the greatest settlement of 64.35mm and U4 recorded the smallest settlement of 0.01mm. Monitoring stations (CP2, CP3, CP4, and CP5) located on the embankment of the dam exhibited more settlement than those off the embankment of the dam. The impounded volume of water in the lake reservoir during the rainy season consequently causes both vertical and lateral (horizontal) movements, which may be the major reason for the large settlement that took place on the monitoring stations located along the embankment of the dam. The movement of 64.35 mm at CP4, which ostensibly amounts to a settlement of about 4.02mm per annum, seems too large and could be ascribed to a sudden instantaneous movement of the rock on which it is placed or merely to tampering. This is because, while the wall continuously experience fairly equal loading along its cross axis, points located on the longitudinal axis are supposed to be subjected to similar forces and hence movements. In addition, from the result of the soil test by Olatunji (1992), it can be concluded that hydrological factors have influenced the vertical movements of the monitoring stations/benchmarks because they are located on clay soil, which expands when wet and contracts when dry.

1976 – 2006 LEVELLING

In February 2006 a levelling campaign, comprising of a network of 25 stations was executed by Anosike (2007). This was the most extensive levelling network around the dam. The height of eight stations, which were observed in the levelling of 1976, were compared to those of 2006 as shown in column 3 of Table 5(a). The results show that the stations CP1, CP4, U4 and BMB were continuously moving downward for 30 years this can and be attributed the weight of the dam. The station CP4 has the highest amount of settlement of 96.44mm and it was noted to have the greatest average absolute displacement of 3.22mm per annum over the last thirty years. An inspection of the site to examine the condition of the monitoring station CP4 revealed that the soil around the station has been eroded and thus, that the station is out of position, due to the erosion around it, the erosion itself is because of rain water.

In addition, from Table 5(a), uplifts occurred at station CP2, CP3, CP5 and U1 in the last 14 years. The movements were in each case less than 1.5mm and can thus be considered insignificant, since some of the differences can be attributed to instrumental and observational errors and the use of reference points that are assumed stable. In view of the general movement of the benchmarks/monitoring stations levelled between 1976 and 2006, it can be concluded that they are fairly stable. Except for the monitoring station CP4 which needs to be protected from further erosion because if this continues it may have serious consequences on the embankment of the dam.

NETWORK B

1976–1991 LEVELLING

The heights of three benchmarks that were observed in the levelling of 1976 and 1991 and the vertical movements that occurred at these benchmarks in the 15-year period are contained in column 1of Table 5(b). These results show that BM9 had an uplift of 5.87mm while BM5 and CP12 had settlements of 14.20mm and 4.32mm, respectively. The cause of the uplift at BM9 can be attributed to the fact that the wall of the building on which it was embedded has attained a level of stability, after it must have settled for some years after construction. The settlement recorded at BM5 can be attributed to the weight of the water tower on which it is embedded. In general, none of the observed movements poses any serious danger to the benchmarks and their host structures or soil.

1976–1992 LEVELLING

The heights of four benchmarks that were observed in the levelling of 1976 and 1992 and the vertical movements that occurred at them are contained in column 2 of Table 5(b). The results show that there was upliftment at benchmark BM11, while other benchmarks recorded settlement. CP12 had the greatest settlement value of 9.06mm. Samples of soil around each benchmark were tested by Sule (1992) and were found to be clay. It is worthy to note that soils containing clay material expand when wet and contacts when dry. Since the observations were taken during the rainy season, this could be responsible for the settlement recorded at the benchmarks (BM5, BM9 and CP12). Also, the uplift recorded at BM11 is as a result of reduction in the rate of consolidation of the soil hosting the building in which the benchmark is embedded. Generally, all movements recorded between 1976 and 1992 levelling can be considered as insignificant and as such benchmarks are not posed to any serious danger.

1976–1993 LEVELLING

The results of 1976 against 1993 levelling show that movements had occurred on all the benchmarks that were tested, as shown in Table 5(b). The four benchmarks were all uplifts. The results show that BM5, BM9, and CP12 moved upward by 3.52mm, 0.22mm and 4.37mm, respectively, when compared to the levelling of 1992. These movements signify the instability of these benchmarks and are attributed to the increase in human and vehicular activities, vibration caused by heavy machinery which were close to the benchmark in 2000. Another factor that must have affected BM5 is the reduction in the load on the benchmark due to non-usage of the water tank on the tower of which it is embedded.

1976– 2002 LEVELLING

The results of 1976 to 2002 levelling (Table 5(b)) indicated that uplift occurred at five benchmarks (BM5, BM9, BM10, BM11, CP13) and settlement at the other two benchmarks (BM18, CP12). BM9 exhibited upward displacement of 6.35mm between

1976 and 2002; it can be said that the building hosting the benchmark is resting on a stable foundation. The variation in movements can be attributed to seasonal changes in soil condition and observational errors. In addition, CP13 has shown an upward displacement of 8.03mm from 1976 to 2002. This can be attributed to growth of the roots of trees found about 0.8m close to it. CP12 recorded the greatest settlement of 18.91mm between 1976 and 2002. This movement can be attributed to the collapsible nature of the soil around the benchmark, as visible sign of the sinking of the soil can be seen.

BM5 exhibited a downward displacement of 15.29mm between 1976 and 1992, 11.77mm between 1976 and 1993, 5.58mm between 1976 and 2002. These show a reduction in the rate of settlement. These movements can be attributed to reduction of the load on the benchmark due to the non-usage of the water tank on the tower and possibly vibration caused by heavy machinery used at a construction rite nearby in 2000.

Finally, it can be concluded that the majority of the benchmarks show upward displacements with a maximum of 8.03mm.

This shows clearly that there was a reduction of settlement of large structures for the years under review. In addition, from the result of the soil test by Sule (1992), it was obvious that hydrological factors have some influence on the vertical movements because the benchmarks are located on clay soil, which expands when wet, and contracts when dry.

Concluding Remarks

This paper has examined the behaviour of some benchmarks on the main campus of Ahmadu Bello University, Zaria, and reported the deformation studies of the area around the University's dam. From this study, it was found that soil erosion, the self-weight of the dam and variations in water pressure (resulting from the annual variation in water volume) and stability problems resulting from high pore pressures are responsible for instabilities along embankment of dams. In addition, in this study, it has been found that there was a reduction for settlement of large structures that hosted some benchmarks for the years reviewed and that benchmarks located upland were greatly affected by hydrological factors and human activities.

Finally, it is recommended that the identification and removal of unstable reference points be the focus of future research in the university and that benchmarks should be placed where danger of destruction is minimal.

REFERENCES

Aladewolu, J. D. (1991). *A Study of the Stability of Some Wall-mounted Benchmarks on A.B.U. Main Campus*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaira.

Amand, A., Adam, N., Hassen, R., Inglada, J., Duru, J., Closa, J., and Einedar, M. (2003). ASAR ERS interferometic phase continuity. *International Geo science and Remote Sensing Symposium*, 21-25 July, 2003, Toulouse (France).

Anosike, U. N. (2007). *Deformation Monitoring of Ahmadu Bello University Dam by Geodetic Levelling*. BSc Thesis, Department of Surveying and Geomatics Engineering, Ahmadu Bello University, Zaira.

Arinola, L. L. (1976). *Levelling Control Network in Ahmadu Bello University, Zaria*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaira.

Celik, R. N., Ayan, T., Denli H., Ozludemir, T., Erol, S., Ozoner, B., Apaydn, N., Erincer, M., Leinene, S., Groten, E. (2001). Monitoring Deformation on Karasu Viaduct Using GPS and Precise Levelling Techniques, *Kluwer Academic Publisher*, Netherlands, pp. 35-41.

Clarke, D. (1978). *Plane and Geodetic Surveying for Engineers*, Vols 1 and 2, 6th edition, Constable and Co Ltd, Great Britain.

Ebong, M. B. (1981). *A Study and Analysis of the Geodetic Levelling of Nigeria.* PhD Thesis, Department of Surveying, University of Newcastle upon Tyne, United Kingdom.

Ebong, M. B. and Musa, A. A. (1992). Benchmark Stability in Nigeria. *Survey Review,* Vol. 31, No. 243, pp. 263-278.

Featherstone, W. E., Denith, M. C., Kriby, J. F. (1998). Strategies for the accurate determination of orthometric heights from GPS. *Survey Review,* Vol. 34, No. 267, pp. 278-296.

Gabriel, A. K, Goldstein, R. M., Zebken, H. A. (1989). Mapping small elevation changes over large areas: Differential radar interferometry. *J. Geophs. Res.,* 94 (B7), pp.9183- 9191.

Higgins, B. M. (1999). *Heighting with GPS*: *Possibilities and Limitations*. In Proceedings of

Geodesy and Surveying in the Future, National Land Survey of Sweden, Gaile, Sweden.

Isa, S. M. (1978). *Large Scale Survey of Kubanni Dam*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaria.

Musa, A. A. (1992). *A FORTRAN Suite for the Adjustment and Analysis of Levelling Networks*. Technical Report, Department of Surveying, Ahmadu Bello University, Zaria.

Ojigi, M, L. (1993). *Z-component Deformation Measurement of Structural and Ground Subsidence in Part of A.B.U. Main Campus*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaria.

Olatunji, O. S. (1992). *Subsidence Survey at Ahmadu Bello University Dam Site*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaria.

Ologe, K. O. (1973). The Kubanni Dam. *Savanna Journal, Vol*. 2, No. 1, pp 45-49.

Oluyori, P. D. (1987). *A Subsidence Monitoring Program for Ahmadu Bello University Dam, Zaria*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaria.

Onuche, O. M. (2002). *Vertical Movement of Benchmarks on the Main Campus of Ahmadu Bello University, Zaria*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaria.

Richardus, G. (1977). *Project Surveying*. North Holland Publishing Co., Amsterdam, New York.

Soile, S. A. (1979). *Extension of Level Network on the Main Campus of Ahmadu Bello University, Zaria*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaria.

Sule, J. O. (1992). *Structural and Ground Subsidence on Ahmadu Bello University, Main Campus*. BSc Thesis, Department of Surveying, Ahmadu Bello University, Zaria.

SURCON (2003). *Specifications for Geodetic Surveys in Nigeria*. Surveyors' Council of Nigeria, Lagos, Nigeria.

US Army Corps of Engineers (2002). *Structural Deformation Surveying* (EM 1110-21009), US Army Corps of Engineers, Washington, DC.

Verdonck, D. (2004). Determination of vertical surface deformation from repeated historical levelling in Cascada. *American Geophysical Union, Spring Meeting, United States*.AGUSM.G21A.03V.