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On The Densification Of Khartoum Geodetic Control Network

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Abstract:

The Department of Surveying Engineering- University of Khartoum, with close cooperation with the General Survey Directorate (GSD), of Khartoum State, densified the existing Khartoum State Geodetic network. This paper outlined the observation, computation work, and analysis carried out for the network densification. The densified network was established to a very high degree of accuracy and reliability to be accepted as reference geodetic stations. The paper outlined the network densification methodology and the quality control of the obtained densified geodetic control points coordinates on ITRF2000.

Key words: GSD, GNSS, TBC, GCP, CTP, WGS84, VLBI, ITRF2000, QC, RINEX

1. Introduction

The establishment of geodetic networks can be considered as one of the primary tasks of geodesy. The geodetic networks, which may be of a local or regional nature, or even of global extent, have a variety of uses in the realms of both scientific and applied geodesy, in order to permit the many and varied surveying, mapping and charting programs to be referenced to some common reference system. It is necessary to have a common reference framework of geodetic control points [8, 10]. An accurate framework can be useful for various levels of users, whose always require a reliable coordinate reference system as a basic requirement for the successful execution of all kind of survey operations and related projects. The definition and densification of coordinate reference systems hinge on points whose 2D or 3D positions are known to be of a high degree of accuracy. Nowadays, for accurate and efficient policy decisions to be taken, the development and utilization of Geographic Information System whose accuracy and effectiveness is dependent on a suitable Geodetic Control Network. The development of satellite technology, especially its application in geodesy through the use of GNSS has opened a new dimension in the observation and strengthening of Geodetic Control Networks worldwide.

The main aim of this paper was to outline the densification of Khartoum geodetic network of control points by using GNSS for the establishment and provision of coordinates for the densified control points through proper connectivity with the existing geodetic control points based on ITRF2000. As part of the geodetic survey activities, the Department of Surveying Engineering team undertook the reconnaissance for the establishment of 10 new Ground Control Points (GCP) and by defining operations based on Khartoum State reference stations, describing the activities for GNSS observations, processing and adjustment. The GNSS observation has been carried out by the survey team and the processing has been carried out by means of Trimble Business Center (TBC) in the Department premises at the University of Khartoum. All newly established GCPs were reduced and tied to the existing Khartoum State reference stations. These additional control stations were selected with the consideration to the accuracy requirements related to GNSS base stations, rovers and control network coverage. This help in the reduction of all GNSS expected errors. The final GCP results were classified as cartesian, geographic and UTM coordinates for each ground control station associated with their corresponding ellipsoidal heights in ITRF2000.0 and standard errors. The control points accept-reject criteria were based on the data and documentation which has been subjected to review and possible field checks. Noncompliance with the GCP product specifications require full or partial re-work depending on the nature of the deficiency. The same is the case for non-compliance with procedure specifications, unless it can be documented that the product still complies with the product specification.

2. Reference Coordinate Systems

In determining the ground control points on Earth from satellite observations, three different reference coordinate systems are important. First of all, satellite positions at the instant of observation are specified in the "space-related" satellite reference coordinate systems [3, 6, 7]. These are three-dimensional rectangular systems defined by the satellite orbits. Satellite positions are then transformed into a three-dimensional rectangular geocentric coordinate system, which is physically related to the Earth. As a result of satellite positioning observations, the positions of the 10 new points are determined in this coordinate system. Finally, the geocentric coordinates are transformed into the more commonly used and locally oriented geodetic coordinate system. The following subsections describe these three coordinate systems. Figure.1 below illustrates a satellite reference coordinate system, (X_s , Y_s, Z_s) [6], the reference axis X_s, the origin of the satellite coordinate system is at *G*; the Y_s axis is in the mean orbital plane; and Z_s is perpendicular to this plane. A satellite at position S_1 would have coordinates X_s, Y_s, & Z_s, as shown in Figure.1. For any instant of time, the satellite's position in its orbit can be calculated from its orbital parameters [7], which are part of the broadcast ephemeris.

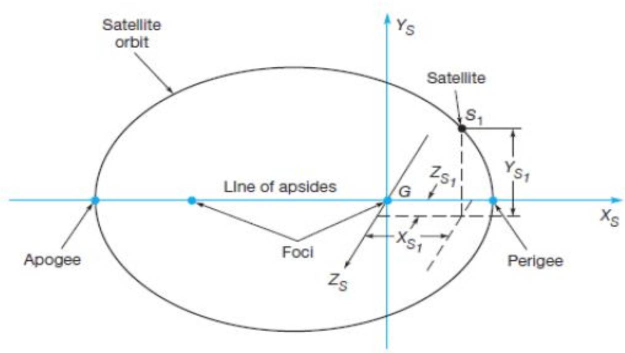

Figure.1: satellite reference coordinate system

2.1 The Geocentric Coordinate System

As usual, the objective of satellite surveys is to locate points on the surface of the Earth, it is necessary to have a so-called terrestrial frame of reference, which enables and relating points physically to the Earth. The frame of reference used for this, is the geocentric coordinate system (X_e , Y_e , Z_e). This three-dimensional rectangular coordinate system has its origin at the mass center of the Earth. Its X_e axis passes through the Greenwich meridian in the plane of the equator, and its Z_e axis coincides with the Conventional Terrestrial Pole (CTP) [8]. To make the conversion from the satellite reference coordinate system to the geocentric system, four angular parameters are required which define the relationship between the satellite's orbital coordinate system, key reference planes and lines on the Earth [12]. These four parameters are (1) the *inclination* angle, i, (2) the *argument of perigee*, ω, (3) the *right ascension of the ascending node*, Ω, and (4) the *Greenwich hour angle of the vernal equinox [7,13]*. These parameters are known in real time for each satellite based upon predictive mathematical modeling of the orbits. Where higher accuracy is needed, satellite coordinates in the geocentric system for specific epochs of time are determined from observations at the tracking stations using precise ephemerides [11].

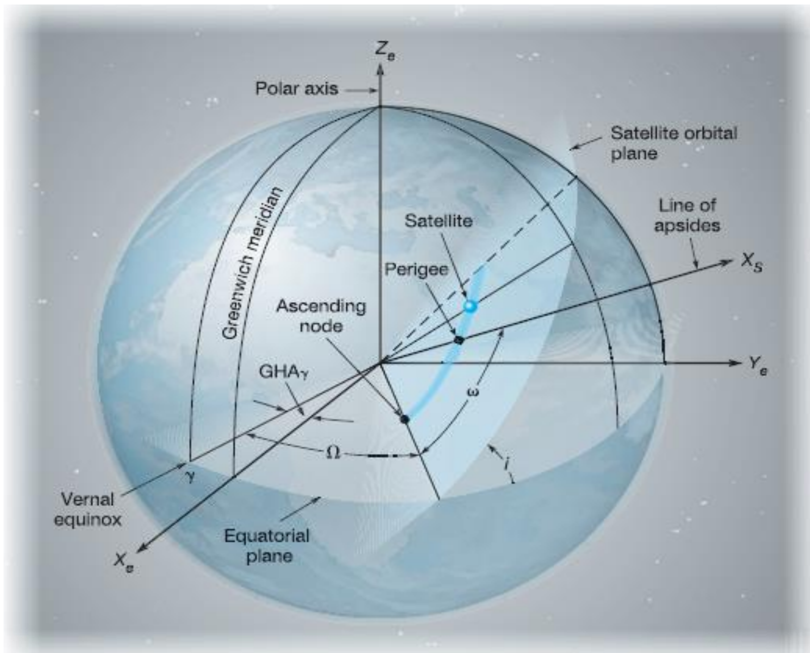

Figure.2: Parameters involved in transforming from the satellite reference coordinate system to the geocentric coordinate system.

2.2 The Geodetic Coordinate System

Although the positions of points in a satellite survey are computed in the geocentric coordinate system described in the preceding subsection, in that form they are inconvenient for use by positioning users including surveyors. Figure.3 illustrate that, the geodetic coordinate system with their: (1) origin at the Earth's center, geocentric coordinates are typically extremely large values, (2) the X-Y plane in the plane of the equator, the axes are unrelated to the conventional directions of north-south or east-west on the surface of the Earth, and (3) geocentric coordinates give no indication about relative elevations between points. For these reasons, the *geocentric* coordinates are converted to *geodetic* coordinates of latitude (ø), longitude (λ) and height (*h*) so that reported point positions become more meaningful and convenient for users.

Conversions from geocentric to geodetic coordinates, and vice versa are readily made (equations, 1,2 and 3). From figure.3, it can be shown that, geocentric coordinates of point *P* can be computed from its geodetic coordinates using the following equations:

$$X_{\mathrm{P}} = \left(\mathrm{R}_{N_P} + h_P\right) cos\emptyset_P \, cos\lambda_P \quad \text{……………………} \; (1)$$

$$Y_{\mathrm{P}} = \left(\mathrm{R}_{N_P} + h_P\right) cos\emptyset_P \, sin\lambda_P \quad \text{……………………} \; (2)$$

$$Z_{\mathrm{P}} = \left[\mathrm{R}_{N_P}\left(1 - \mathrm{e}^2\right) + h_P\right] sin\emptyset_P \quad \text{……………………} \; (3)$$

Where $\mathrm{R}_{N_P} = \frac{a}{\sqrt{1-\mathrm{e}^2 sin^2\emptyset_P}}$

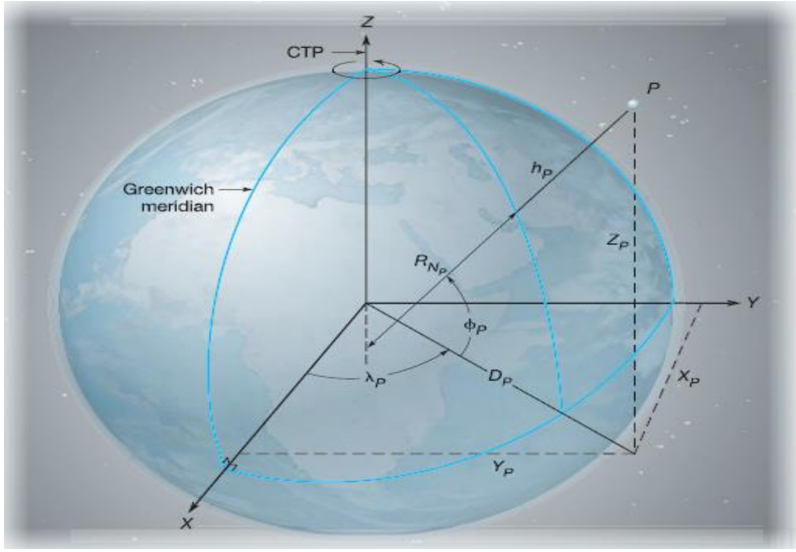

Figure.3: Relationship between geodetic and geocentric coordinate systems.

In the above equations, X_{P}, Y_{P} & Z_{P} are the geocentric coordinates of any point P, and the term e is the *eccentricity* of the reference ellipsoid. The $\mathrm{R}_{\mathrm{N_P}}$ is the radius in the prime vertical of the ellipsoid at point P, and a is the semi major axis of the ellipsoid. In equations (1. 2,3), the north latitudes are considered positive and south latitudes negative. Similarly, sometimes east longitudes are considered positive and west longitudes negative [7,13].

2.3 Global datum

The WGS84 was established from the coordinate position of about 1600 points around the globe, fixed largely by TRANSIT satellite observations [11]. At the present time, its origin is geocentric (i.e. the center of mass of the whole Earth) and its axes virtually coincide with the International Reference Pole and International Reference Meridian. Designed to best fit the global geoid as a whole means it does not fit many of the local ellipsoids in use by many countries. In Sudan, for instance, it lies about ±6 m, above and below the geoid and slopes nearly from the center of the country to the north (positive) and to the south (negative), resulting in the geoid–ellipsoid separation being between ±6m in the entire territory of Sudan. It is also worth noting that the axes are stationary with respect to the *average* motions of the dynamically changing earth [1, 5]. For instance, tectonic plate movement causes continents to move relative to each other by about 10 cm per year [13]. Local movements caused by tides, pressure weather systems, etc., can result in movement of several centimeters. The result is that the WGS84 datum appears to move relative to the various countries [7].

Another global datum almost identical to the WGS84 Reference System is the International Terrestrial Reference Frame (ITRF) produced by the International Earth Rotation Service (IERS) in Paris, France. The system was produced from the positional coordinates of over 500 stations throughout the world [13], fixed by a variety of geodetic space positioning techniques such as Satellite Laser Ranging (SLR), Very Long Baseline Interferometry (VLBI), Lunar Laser Ranging (LLR), Doppler Ranging Integrated on Satellite (DORIS) and GPS. Combined with the constant monitoring of earth rotation, crustal plate movement and polar motion, the IERS have established a very precise terrestrial reference frame [7,8], the latest version of which is the

ITRF2020s. This TRF comprises a list of Cartesian coordinates (*X*, *Y*, *Z*), with the change in position (dX, dY, dZ) in meters per year for each station. The ITRF2000 is available as a RINEX format text file from the IERS website. The ITRF is the most accurate global TRF and for all purposes is identical to the WGS84 TRF. The ITRF2000 was based on the time series of station positions and earth orientation parameters using GPS observations computed in the epoch ITRF2000.0 [13].

3. Fundamental Aspects

3.1 General

Overall basis for the geodetic survey work involved is the Specification on “The Establishment of Geodetic Ground Control Points”, the densification of the established Ground Control Points comprising of 10 stations distributed throughout Khartoum, Omdurman and Bahri areas (Figure.8). These stations were tied to the well-distributed Khartoum geodetic control reference stations in ITRF2000. The survey team, carried out a reconnaissance exercise in order to inspect the existing Khartoum stations that could possibly be used for the densification of the network. The GCPs were established using simultaneous observations using a minimum number of two base stations. The GNSS campaign resulted in the successful observation of about 60 minutes common data sessions at the base stations. The implementation of the adopted instrument setting parameters and the procedures set for field observations and processing was used for the quality assurance.

According to the technical specifications all GNSS observations were processed in the Department geodetic laboratory; all processing was performed on the TBC software package. The computations for the GCPs utilized the information disseminated by the survey team and referenced to the ITRF2000 frame. Likewise other parameters were utilized in processing, then the results are checked and analyzed to maintain the quality control of the results.

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3.2 Reliability of Khartoum State Reference Stations

Based on the scope of the survey work, the densified ground control stations were tied to at least two (2) well distributed geodetic base stations, in order to increase the reliability proposed in the implementation plan to tie the GNSS measurements to base stations. In such a way that to maintain the best loop geometry and that, only base stations which are located in the vicinity of the area are used.

Special consideration was being taken by the survey team for checking the reliability of Khartoum State reference stations used in this survey. The team performed calibration checks and reliability tests, before carrying out any geodetic survey activity. The instrument went for the company calibration and passed all calibration checks and indicated no errors or defects.

To meet the survey requirements, the coordinates of the adopted Khartoum State ground control stations were also assessed for the reliability of their stability by comparing their known coordinates with the ones obtained from the survey observations. The survey team, confirmed the stability of the base GCPs used, and found to be in good condition and suitable for the purpose of the densification survey works.

3.3. The Quality Assurance specifications and Quality Control

In order to obtain a further indication on the quality of receivers’ recordings at each GCPs, the specifications for the quality assurance were set out in all GNSS receivers used. the observation data of several stations were systematically analysed and adjusted. This led to the possibility of estimating the quality of the reference observations and the resulting co-ordinates within the ITR2000. Another advantage of this, was the possibility of determining the data quality of the potential stations of the GCPs linked to the Khartoum State geodetic network.

Due to the fact that all base lines used are relatively short, during the period of GCPs observations, so the ionospheric disturbances were reduced based on the combination of L1 and L2 observations. The parameter estimation itself has been carried out with a linear combination of the L1 and L2 observations. The short baselines used all through the survey observations activities, ease to solve the phase ambiguities.

Table.1 illustrates, an overview of quality assurance specifications used for the analysis of GCPs stations.

Table.1: Overview of parameters used for GCPs processing and analysis

Parameter	Test Campaign of ITRF Stations
Observation Time:	2019
Reference GCPs	Khartoum State reference stations
Datum:	ITRF2000, Epoch 2000.
Software:	Trimble Business Center
Frequency:	L1 and L2
Elevation mask:	10 degrees
Troposphere parameters:	Hopfield tropospheric model
Ambiguity resolution:	Quasi-Ionosphere-Free Strategy
Minimum one-hour GNSS static observation	

The GNSS quality control (QC) needed to check the quality of GNSS observations before post-processing, so that, the precision of GNSS data analysis can be improved. The GNSS_QC was designed to calculate the quality control (QC) parameters such as data gaps, cycle slips, low elevation angle, ionospheric delay, multi-path effects and DOP etc. during the period of GNSS observation. It can be used to read and calculate the QC parameters from RINEX files. The TBC software gives users brief statistics, time series plots and graphs of QC parameters. The GNSS_QC can simply be performed, together with the quality checking of GNSS data that was difficult for surveyors in the field.

3.4 GPS Antenna Heights

One of the most important activities in the GNSS field survey is the careful measurement and documentation of the antenna heights. Usually, it is very difficult to detect and correct an erroneous antenna height during post-processing. Therefore, the survey team used a rigid control system during the GNSS campaign in order to set the ground for a smooth processing of GNSS data. For all the GCP points the antenna heights were measured true vertical, so no reduction of slant heights was necessary.

The height values were measured between the surface of the brass plate of the observation monument and the bottom of the ground plane of the antenna. To obtain the correct link between the electronic phase centre inside the antenna and a physical reference point outside of the antenna, an antenna phase centre model has been used. For this purpose, the phase centre models of the IGS [3] respectively of the National Geodetic Survey (NGS) were applied. This antenna phase model fits to the antenna reference point, which is at the bottom of the preamplifier. The finally used antenna heights were measured in the field by the survey team.

4. GPS Observables

4.1 pseudoranges Observables

The basic GNSS observables are code pseudoranges and carrier phases as well as Doppler measurements. The principle of the GNSS measurements and their mathematical expressions are described. The pseudorange is a measure of the distance between the satellite and the receiver's antenna. The distance is measured through measuring the GNSS signal transmitting time from the satellite to the GNSS receiver's antenna. The transmitting time is measured through maximum correlation analysis of the receiver code and the GNSS signal [3]. The receiver code is derived from the clock used in the GNSS receiver. The measured pseudorange is different from the geometric distance between the satellite and the receiver's antenna because of the errors of both clocks and the influences of the signal transmitting mediums. It is also notable that the path of the signal transmission differs slightly from the geometric path. The transmitting medium not only delays the transmitting of the signal, but also bends the transmitting path of the signal [2, 4]. The GNSS signal emission time of the satellite is denoted by $\mathrm{t_e}$, and the GNSS signal reception time of the receiver is denoted by $\mathrm{t_r}$. In case of vacuum medium and error-free situation, the measured pseudorange is equal to the geometric distance and can be presented by:

$$\mathrm{R_r^S}(\mathrm{t_r}, \mathrm{t_e}) = (\mathrm{t_r} - \mathrm{t_e})c \qquad \text{......................} \ (4)$$

where *c* denotes the speed of light, and subscripts r and s denote the receiver and satellite, respectively. On the left-hand side, $\mathrm{t_r}$ denotes the epoch at which the pseudorange is measured. $\mathrm{t_e}$ and $\mathrm{t_r}$ as indicated above, are considered true emission time and reception time of the GNSS signal. Taking both the satellite and receiver clock errors into account, the pseudorange can be represented as [7]: -

$$\mathrm{R_r^S}(\mathrm{t_r}, \mathrm{t_e}) = (\mathrm{t_r} - \mathrm{t_e})\mathrm{c} - (\delta\mathrm{t_r} - \delta\mathrm{t_s})\mathrm{c} \quad \text{......} \ (5)$$

where $\delta\mathrm{t_r}$ and $\delta\mathrm{t_s}$ denote the clock errors of the receiver and satellite, respectively. The GNSS satellite clock error term $\delta\mathrm{t_s}$ is known through GNSS satellite orbit determination. The clock errors are usually modelled by polynomials of time [6]. The constant term represents the bias and the linear term the drift of the clocks. These coefficients are transmitted along with the navigation message to the users [5]. More precisely, the satellite clock error corrections can be also obtained from all IGS data centers (cf., e.g., www.gfz-potsdam.de). They are determined along with the precise IGS orbits and have higher resolution in time. The geometric distance of the first term on the right-hand side of Equation (5) is given by: -

$$\rho_r^S(\mathrm{t_r}, \mathrm{t_e}) = \sqrt{(x_s - x_r)^2 + (y_s - y_r)^2 + (z_s - z_r)^2} \quad \text{........} \ (6)$$

where the satellite coordinate vector (x_s, y_s, z_s) is a vector function of the time $\mathrm{t_e}$, and the receiver coordinate (x_r, y_r, z_r) is a function of the time $\mathrm{t_r}$.

4.2 Carrier Phases

The carrier phase is a measure of the phase of the received satellite signal relative to the receiver-generated carrier phase at the reception time. The measurement is made by shifting the receiver-generated phase to track the received phase [5, 7]. The number of full carrier waves between the receiver and the satellite cannot be accounted for at the initial signal acquisition. Therefore, measuring the carrier phase is to measure the fractional phase and to keep track of changes in the cycles. The carrier phase observable is an accumulated carrier phase observation. The fractional carrier phase can be measured by electronics with precision better than 1% of the wavelength, which corresponds to millimeter precision [5]. This is also the reason why the phase measurement is more precise than that of the code. A full carrier wave is called a cycle, and the ambiguous integer number of cycles in the carrier phase measurement is called ambiguity. The initial measuring has a correct fractional phase and an arbitrary integer counter setting at the start epoch. Such an arbitrary initial setting will be adjusted to the correct one by modelling with ambiguity parameters [7]. In the case of a vacuum medium and an error-free situation, the measured phase can be presented by [5]:

$$\emptyset_r^s(t_r) = \emptyset_r(t_r) - \emptyset^s(t_r) + N_r^s \quad \text{............... (7)}$$

where subscript r and superscript s denote the receiver and satellite respectively, t_r denotes the GNSS signal reception time of the receiver. $\emptyset_r$ denotes the phase of receiver's oscillator. $\emptyset^s$ denotes the received signal phase of the satellite. N_r^s is the ambiguity related to receiver r and satellite s.

4.3 Differential GPS

The degradation of the point positioning accuracy by SA before two decades ago [7], has led to the development of Differential GPS (DGPS). This technique is based on the use of two (or more) receivers, where one (stationary) reference or based receiver, which is located at a known point and the position of the (mostly moving) remote receiver is to be determined. At least four common satellites must be tracked simultaneously at both sites. The known position reference receiver is used to calculate corrections to the GNSS derived position or to the observed pseudoranges. These corrections are then transmitted via telemetry (i.e., controlled radio link) to the roving receiver and allow the computation of the rover position with far more accuracy than the single-point positioning mode.

The fundamental assumption in Differential GNSS (DGPS) is that the errors within the area of survey would be identical. This assumption is acceptable for most engineering surveying where the areas involved are small compared with the distance to the satellites.

4.4 Relative positioning

The most accurate positions are currently obtained using relative positioning techniques. Similar to both DGPS, this method removes most errors by utilizing the differences in either the code or carrier phase ranges. The objective of relative positioning is to obtain the coordinates of a point relative to another point. This can be mathematically expressed as:

$$\begin{aligned} X_B &= X_A + \Delta X \\ Y_B &= Y_A + \Delta Y \quad \text{............. (8)} \\ Z_B &= Z_A + \Delta Z \end{aligned}$$

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where (X_A, Y_A, Z_A) are the geocentric coordinates at the base station *A*, (X_B, Y_B, Z_B) are the geocentric coordinates at the unknown station *B*, and $(\Delta X, \Delta Y, \Delta Z)$ are the computed baseline vector components (Figure.4). Relative positioning involves the use of two or more receivers simultaneously observing pseudoranges and phase measurements at the endpoints of lines. Simultaneity implies that the receivers are collecting observations at the same time and at the same epoch rate. This rate depends on the purpose of the survey and the final desired accuracy [4, 11].

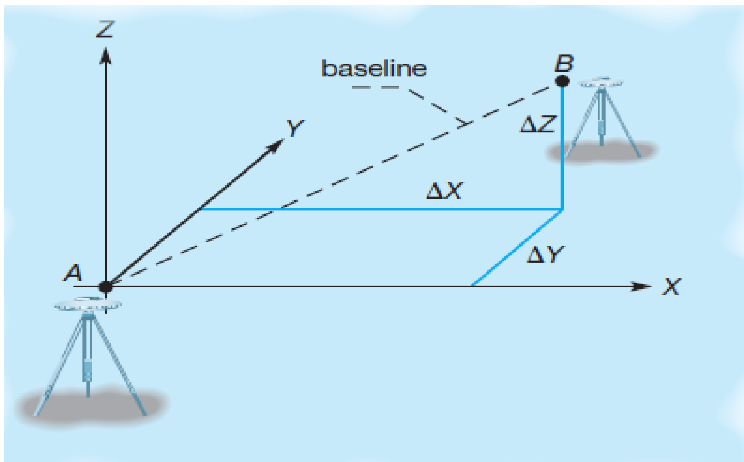

Figure.4: Computed baseline vector components.

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DGPS techniques can be illustrated as follows: -

(a) Single Differencing: as illustrated in Figure.5, single differencing involves subtracting two simultaneous observations made to one satellite from two points. This difference eliminates the satellite clock bias and much of the ionospheric and tropospheric refraction from the solution. It would also eliminate the effects of SA (if it were turned on), the phase equations for the two points are: -

$$\emptyset_A^j(t) - f^j\,\delta^j(t) = \frac{1}{\lambda}\,\rho_A^j(t) + N_A^j - f^j\,\delta_A(t) \;........\; (9)$$

$$\emptyset_B^j(t) - f^j\,\delta^j(t) = \frac{1}{\lambda}\,\rho_B^j(t) + N_B^j - f^j\,\delta_B(t) \;.........\; (10)$$

The difference in these two equations yields

$$\emptyset_{AB}^j(t) = \frac{1}{\lambda}\,\rho_{AB}^j(t) + N_{AB}^j - f^j\,\delta_{AB}(t) \;........................\; (11)$$

(b) Double Differencing: as illustrated in Figure.6, involves taking the difference of two single differences obtained from two satellites *j* and *k*. The procedure eliminates the receiver clock bias. Assume the following two single differences, the differencing equation can be given as:

$$\emptyset_{AB}^j(t) = \frac{1}{\lambda}\,\rho_{AB}^j(t) + N_{AB}^j - f^j\,\delta^j{}_{AB}(t) \;............\; (12)$$

$$\emptyset_{AB}^K(t) = \frac{1}{\lambda}\,\rho_{AB}^K(t) + N_{AB}^K - f^K\,\delta^j{}_{AB}(t) \;.........\; (13)$$

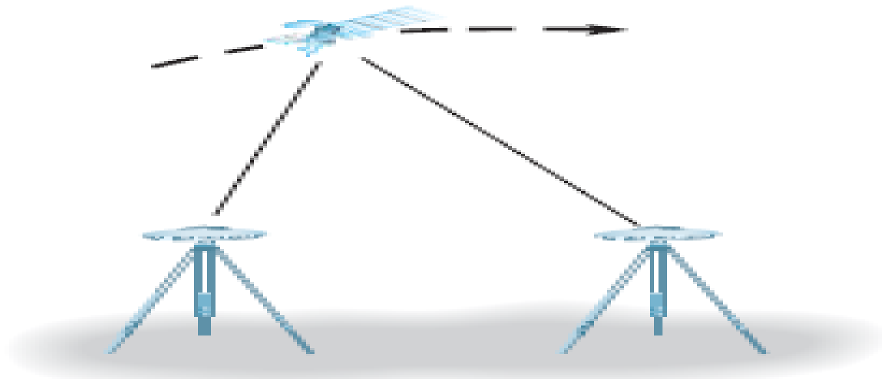

Figure.5: single differencing.

Note that the receiver clock bias will be the same for observations on satellite *j* as it is for satellite *k*. Thus, by taking the difference between these two single differences i.e. equations 12 and 13, the following double difference equation is obtained, in which the receiver clock bias errors $f^{j}\ \delta^{\mathrm{j}}{}_{AB}(t)$ and $f^{K}\ \delta^{\mathrm{j}}{}_{AB}(t)$ are eliminated.

$$\emptyset_{AB}^{\mathrm{jK}}(t) = \frac{1}{\lambda}\ \rho_{AB}^{\mathrm{jK}}(t)+N_{AB}^{\mathrm{jK}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(14)}$$

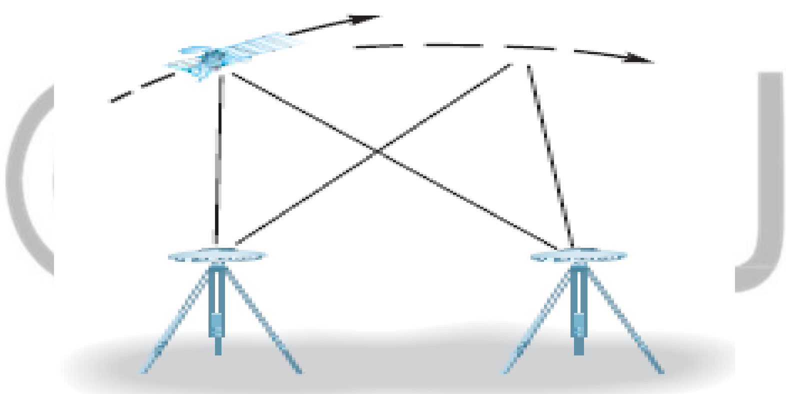

Figure.6: Double differencing.

(c) Triple Differencing: The triple difference illustrated in Figure.7, involves taking the difference between two double differences obtained for two different epochs of time. This difference removes the integer ambiguity from the phase equation, leaving only the differences in the phase-shift observations and the geometric ranges. The two double-difference equations can be expressed as:

$$\emptyset_{AB}^{\mathrm{jK}}(\mathrm{t}_1) = \frac{1}{\lambda}\ \rho_{AB}^{\mathrm{jK}}(t_1) + N_{AB}^{\mathrm{jK}} \ \ldots\ldots\ldots\ldots\ldots \text{(15)}$$

$$\emptyset_{AB}^{\mathrm{jK}}(t_2) = \frac{1}{\lambda}\ \rho_{AB}^{\mathrm{jK}}(t_2) + N_{AB}^{\mathrm{jK}} \ \ldots\ldots\ldots\ldots\ldots \text{(16)}$$

The difference in these two double differences yields the following triple difference equation, in which the integer ambiguities have been removed. The triple difference equation is: -

$$\emptyset_{AB}^{\mathrm{jK}}(\mathrm{t}_{12}) = \frac{1}{\lambda}\ \rho_{AB}^{\mathrm{jK}}(t_{12}) \ \ldots\ldots\ldots\ldots\ldots\ldots \text{(17)}$$

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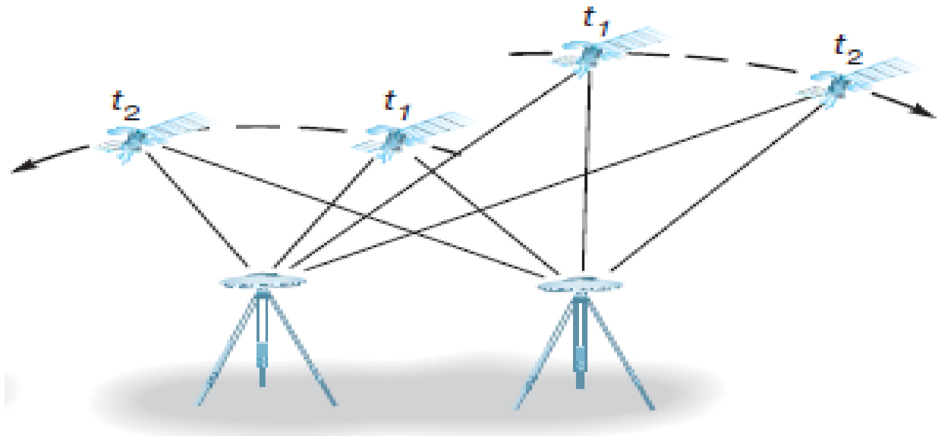

Figure.7: Triple differencing.

The importance of employing the triple difference equation in the solution is that by removing the integer ambiguities, the solution becomes immune to cycle slips. Today's processing software rarely, if ever, uses triple differencing since the integer ambiguities are resolved using more advanced on-the-fly techniques [7, 12].

(c) Static Relative Positioning: - For highest accuracy, for example geodetic control surveys, static surveying procedures are used. In this procedure, two (or more) receivers are employed. The process begins with one receiver (called the base receiver) being located on an existing control station, while the remaining receivers (called the roving receivers) occupy stations with unknown coordinates. For the first observing session, simultaneous observations are made from all stations to four or more satellites for a time period depend on the baseline length. Except for one, all the receivers can be moved upon completion of the first session. The remaining receiver now serves as the base station for the next observation session. It can be selected from any of the receivers used in the first observation session. Upon completion of the second session, the process is repeated until all stations are occupied, and the observed baselines form geometrically closed figures. For checking purposes some repeat baseline observations should be made during the surveying process. The typical epoch rate in static survey is 15 sec. After all observations are completed, data are transferred to a computer for post-processing. Relative accuracies with static relative positioning are about ± (3 to 5 mm + 1 ppm). Typical durations for observing sessions using this technique, with single- and dual-frequency receivers, are shown in Table.2.

Table.2: Typical durations for observing sessions

Method of survey	Single Frequency	Dual frequency
Static	30min. + 3 min/km	20min. + 2 min/km
Rapid Static	20min. + 2 min/km	10min. + 1 min/km

5. Sources of Errors in GNSS Surveys

As is the case in any survey campaign, observations are subject to instrumental, natural, and personal errors. These are summarized as [7]: -

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(a) Instrumental Errors: which include Clock bias, setup error, non-parallelism of the antenna and receiver noise: -

(i) Clock Bias: both the receiver and satellite clocks are subject to errors. The satellite clock bias can be modeled by applying coefficients that are part of the broadcast message. The receiver clock bias can be treated as an unknown, so it should be computed. They can be mathematically removed using differencing techniques for all forms of relative positioning.

(ii) Setup Errors: As with all work involving tripods, the equipment must be in good adjustment, and careful attention should be paid to maintaining tripods that provide solid setups, and tribraches with optical plummets that will center the antennas over the monuments. In GNSS work, tribrach adapters are often used, that allow the rotation of the antenna without removing it from the tribrach. If these adapters are used, they should be inspected for looseness on a regular basis. Because of the many possible errors that can occur when using a standard tripod, special fixed-height tripods and rods are often used. The fixed-height rods can be set up using either a bipod or tripod with a rod on the point. They typically set to a height of precisely 2 m from the antenna reference point (ARP).

(iii) Non-parallelism of the Antennas: Pseudoranges are observed from the phase center of the satellite antenna to the phase center of the receiver antenna. The phase center of the antenna may not be the geometric center of the antenna. Each antenna must be calibrated to determine the phase center offsets for both the $L1$ and $L2$ bands. For antennas, with phase center offsets, the antennas are aligned in the same direction. Generally, they are aligned according to local magnetic north using a compass.

(iv) Receiver Noise: When working properly, the electronics of the receiver will operate within a specified tolerance. Within this tolerance, small variations occur in the generation and processing of the signals that can eventually translate into errors in the pseudorange and carrier-phase observations. Since these errors are not predictable, they are considered as part of the random errors in the system. However, periodic calibration checks and tests of receiver electronics should be made to verify that they are working within acceptable tolerances.

(b) Natural Errors: these include, refraction, relativity and multipathing, as: -

(i) Refraction: Refraction due to the transit of the signal through the atmosphere plays a crucial role in delaying the signal from the satellites. The size of the error can vary from 0 to 10 m. Dual-frequency receivers can mathematically model and remove this error. With single-frequency receivers, this error must be modeled. For surveys involving small areas using relative positioning methods, the majority of this error will be removed by differencing. Since high solar activity affects the amount of refraction in the ionosphere, it is best to avoid these periods.

(ii) Relativity: GNSS satellites orbit the Earth in approximately 12 hours. The speed of the satellites causes their atomic clocks to slow down according to the theories of relativity [3]. The master control station computes corrections for relativity and applies these to the clocks in the satellites.

(iii) Multipathing: Multipathing occurs when the signal emitted by the satellite arrives at the receiver after following more than one path. It is generally caused by reflective surfaces near the receiver. Multipathing can become so great that it will cause the receiver to lose lock on the signal. Many manufacturers use signal filters to reduce the problems of multipathing. However, these filters will not eliminate all occurrences of multipathing, and are susceptible to signals that have been reflected a number of times. Thus, the best approach to reducing this problem is to

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avoid setups near reflective surfaces. Reflective surfaces include flat surfaces such as the sides of building, vehicles, water, and chain link fences.

(c) Personal Errors: include Tripod mis centering error, that directly affect the final accuracy of the coordinates. To minimize it, check the setup carefully before data collection begins and again after it is completed.

6. Planning GPS Observations and Processing

The Department of surveying engineering survey team, adopted Global Positioning System (GPS) Survey Specifications, described the methods and procedures needed to attain a desired survey accuracy standard and precision which are required for the densification of the geodetic control points. The specifications for Post Processed GNSS Surveys shall be based on International adopted standards [3]. The following specifications set forth the minimum requirements that must be met by the survey team when planning for the GNSS (Global Navigation satellite Systems). The precision of the GNSS vector base line results depends on the number of satellites visible simultaneously from each station during an observing session [7]. It is depended on many other factors like satellite geometric relationships, duration of observation, number of satellites observed simultaneously, the uncorrected effects of ionospheric and tropospheric refraction, and the length of base line. The number of possible observing sessions per observing day is a function of the required survey accuracy, satellite availability, and the logistical considerations such as travel and set up time required between observing sessions. To carry out GNSS surveys the following should be carefully considered to meet the required accuracy [9, 11, 12]:

(a) Equipment: The GNSS surveying equipment are generally consists of two major components: the receiver and the antenna. The receiver's requirements are based on their capability in data acquisition and recording, and the software used should meet the survey requirement. Dual frequency receivers are used for observing baselines by using both GPS/GlONAS satellites available. Two types of GNSS instruments were used in the observation campaign, namely, seven Trimble and 6 Geo max zenith20/10 instruments, all operating on the L1 and L2 frequencies.

(b) Miscellaneous Equipment Requirements: All equipment's and devices shall be properly maintained and regularly checked for accuracy. Errors due to poorly maintained equipment/device have been eliminated to ensure that, the survey results meet the survey accuracies, by testing loop closures (figure.9) and observations at known stations.

(c) Redundancy: GNSS control points were designed with sufficient redundancy to detect and isolate blunders and/or systematic errors. Redundancy of control design is achieved by: Connecting each control station with at least two independent baselines Series of interconnecting, closed loops Repeat baseline measurements was used (Figures.9 and 10).

(d) Reference Stations: Khartoum State geodetic control stations were used. To meet the survey accuracies, all GNSS observations were based on ITRF2000 and all existing GCPs were previously linked to IGS stations epoch 2000.0.

(e) Satellite Geometry: In planning the GNSS survey, the following Satellite geometry factors were considered [4, 10]:

- Minimum Number of satellites available at time of observation, (Minimum of 5 satellite).

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- Minimum satellite elevation angle; (Minimum of 10 degrees above horizon).
- Obstructions limiting satellite visibility (clear sky view)
- Positional Dilution of Precision (PDOP) minimum 5.

6.1 Site selection

The existing control stations used to reference and adjust this network is considered to be referenced to the international terrestrial reference frame (ITRF 2000). The known GCPs, hereafter denoted by kss8, kss7, kss6, kss5, kss4 (figures 8 and 10).

For geodetic control densification, ten GCPs locations were selected by using google earth in different locations inside Khartoum state (figure.8). The exploratory visit for selecting GCPs locations were accessed by using the GPS navigator, according to the exploratory visiting surveyors, points positions (10 points) were selected considering the GNSS positioning limiting factors, such as multipath, and the requirement of open sky, safe location, etc. Figure.8, illustrates the locations of the selected sites and the ground control points.

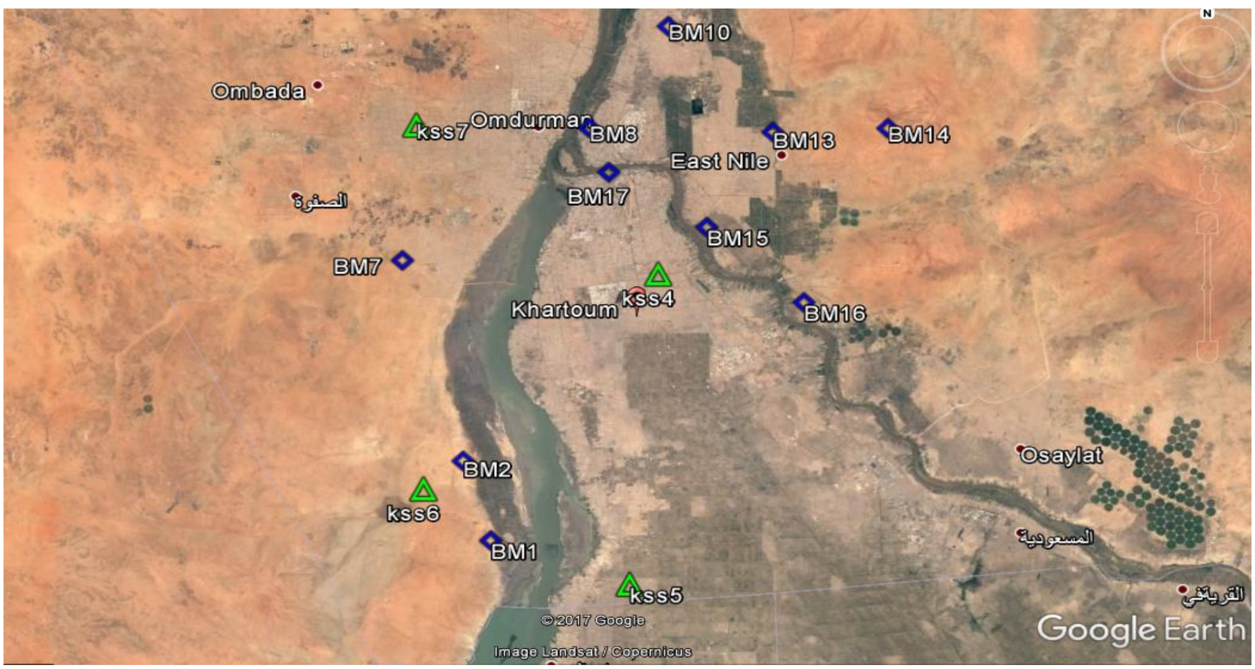

Figure.8: Illustrates the distribution of the selected densification geodetic control points

6.2 Densification Network Design

The survey team has designed the densification network, five known control points and ten new control points. The network was designed with the consideration of the distance between points their distribution, mask angles greater than 10 degrees above the horizontal plane to avoid multipath errors. As well the points locations should be easily accessed, and to guarantee their existence for a long period of time, without any expected obstruction or damage in the future,

6.3 GCPs Monumentation:

The Survey team, established 10 GCPs, shown in figure.8. The coordinates of the GCPs, as mentioned, were based on Khartoum State geodetic network frame. These GCPs control points were verified and found to be acceptable, meeting the survey positional accuracy requirements. The control point specifications are concrete cemented cylindrical pillars. Pillars dimensions are 1m length, 15cm diameter and drilling depths of 80cm, and 20cm above the ground, all pillars were eventually installed properly vertical.

The most important factor for determining Ground Control station's location is the stability, and the special consideration that given to the following GNSS limitations factors, such as:

- Stations are constructed in locations, which are relatively free from GNSS Signals obstructions i.e. clear view of the sky.
- Locations near strong radio transmissions are avoided because radio frequency transmitters, including cellular phone equipment, may disturb satellite signal reception.
- A void locating stations near large flat surfaces such as buildings, large signs, fences, etc., as satellite signals may be reflected off these surfaces causing multipath errors.

6.4 Field observation and Data processing

As the reconnaissance Surveys were carried out for existing ground control station and site selection for the 10 new points with special consideration to the Station stability; obstructions and multipath. Weather conditions were also considered in such a way that, extreme condition (windy), and during periods of significant weather changes observation are to be avoided.

Table.3: Sample of Typical loop Closure for observing sessions

$KSS4\ BM13\ BM10\ BM8\ \ KSS4$				
Baseline	ΔX	ΔY	ΔZ	Distance
$KSS4 - BM13$	-8222.242	6142.986	12961.337	16532.93069
$BM13 - BM10$	2428.841	-8832.824	9648.541	13304.6004
$BM10 - BM8$	5747.214	-4157.263	-9279.09	11679.67532
$BM8 - KSS4$	46.201	6847.08	-13330.797	14986.48684
Vector sum	0.014	-0.021	-0.009	56503.69325
Resultant closure= 0.02679552201 ppm= 0.474226027				

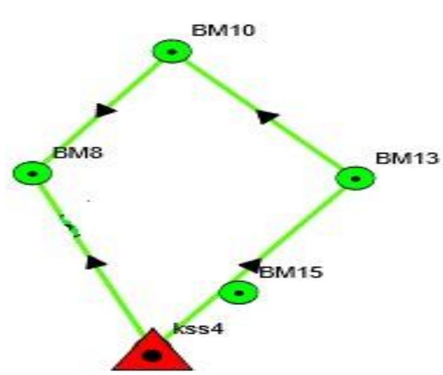

Figure 9: Sample of loop Closure for observing sessions

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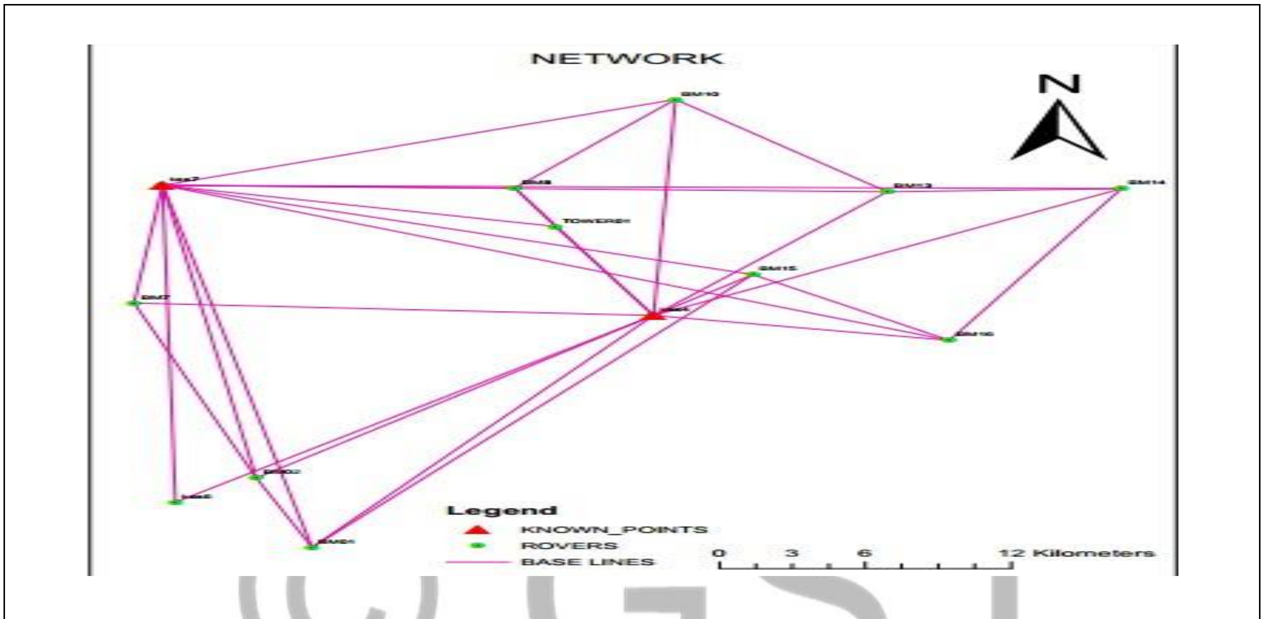
The observation was performed in one session as given in Figure.9, the observation team was divided into three groups (group1 at Omdurman, group2 at Bahri, group3 at Khartoum), and the devices used were prepared to ensure that the batteries were fully charged and the receiver's storage were empty. The tolerances of using elevation mask of 10 degree, PDOP 5 and recording intervals are taken into account.

Figure.10: Illustrates the Densification ground control Network -Field observation



For data processing, observation data were collected and transformed to RINEX format in order to be able to use it in different post processing software environments. Trimble business center (TBC) was used in post processing, baseline observations were processed using a single baseline solution and the broadcast ephemeris. The processing has been done in different ways. The baseline results, consisting of baseline components (ΔX, ΔY and ΔZ) (table.3), the standard deviation of each component and the cofactor matrix elements for each baseline were exported to an excel sheet. The trivial baseline for a session was processed and exported separately to be used for network pre-adjustment data analyses, the result of data processing was verified and quality controlled.

The collected raw GNSS observations were post processed in the office and their results were reviewed and analyzed. The analyzed results are: Baseline processing results; Loop closures (table.3). Results from the least-squares adjustments; post-processing software summary was obtained (table.4 as an example). As well as, the Loop Closure and Repeat Baseline Analysis (Loop closures and differences in repeat baselines) were computed to check for blunders and to obtain initial estimates of the interns' consistency of the GNSS surveys. The accuracy standards for GNSS survey accuracies, of 2.5cm for both horizontal and vertical GCPs were achieved. An example of the base line configuration is given in figure.4. Configurations with different baselines were used for the several sessions, with "n" stations involved only "n-1" baselines can be defined for each session. During the phase of pre-analysis, a strategy was made to solve the ambiguities, and for the short baselines it was possible to solve the ambiguities.

Table.4: Sample of Typical residuals at the densified GCPs obtained from the Adjustment

From	TO	V_x	V_y	V_z
Kss7	BM7	0.015175	0.017156	0.001721
	BM02	0.012928	0.015629	0.002972
	BM01	0.015252	0.018437	0.005355
	BM15	0.017169	0.020520	-0.00438
	BM16	0.014627	0.013875	0.003186
	BM14	0.011195	0.012908	0.001509
	BM13	0.013371	0.014349	0.003667
	BM10	0.011611	0.015015	0.002908
	BM8	0.010521	0.016059	0.000728
	TOWER01	0.013684	0.022204	0.005930
	Kss6	0.015130	0.017299	0.006585

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7. Final Adjusted Coordinates of GCPs Stations

In order to obtain a further indication on the quality of the densified geodetic stations of the Khartoum State, the observation data of well-known ITRF2000 stations and the new stations were systematically analyzed and adjusted. This led to the possibility of estimating the quality of observations on reference stations as well as the densified stations. Another advantage of the study is the possibility of determining the data quality of the potential stations to be used for the link of the densified stations to the global network as given in references [9, 10].

The co-ordinates of the control stations were adjusted by constraining the known stations [5]. The results achieved are shown as repeatability of the individual session solutions, which provides an indication on the accuracy of the network. These repeatability values are generally more realistic when judging the quality of the results since the covariance matrix tends to be too optimistic. The repeatability (standard of individual session solutions and combined final solution) for the stations results are shown in table.5 and found to be approximately 1cm for X and Y, and Z components.

The obtained residuals are found to be about 1cm (table 4). This indicates that there is no deformation existing within the network while fixing the control points. Since the different adjustment strategies resulted in only very small co-ordinate differences, it is recommended that the results of the network adjustment are to be accepted as the final result. Here, it can be summarized that the accuracy of the densified network is better than 0.6cm in longitude and latitude, and better than 1 cm in its height component. The overall accuracy of the densified network with reference to the known geodetic control station in Khartoum can be interpreted as approximately 0.5cm in latitude and longitude, and approximately 1cm in its height component.

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Table.5 and table.7 show the final adjusted cartesian and projected coordinates of the established GCPs for the densification geodetic Control network. Final GNSS data processing and adjustment was performed with the Trimble Business Center software (table.6) to illustrate the minimum, maximum and mean standard errors. The network analysis was divided into three parts: pre-analysis, parameter estimation, and final adjustment.

Table.5: Cartesian Coordinates Adjustment and their standard deviation

Station	X(m)	Y(m)	Z(m)	σx ± (m)	σy ± (m)	σz ± (m)
BM7	5191336.2511	3291669.1197	1697387.9299	0.0100	0.0117	0.0126
BM02	5192874.7528	3298673.5573	1679087.2739	0.0102	0.0100	0.0099
BM01	5193316.8160	3301693.6528	1671819.2933	0.0090	0.0086	0.0087
BM15	5176940.0949	3312686.3927	1700423.6054	0.0091	0.0101	0.0089
BM16	5174171.7802	3320547.9795	1693557.7904	0.0101	0.0085	0.0093
BM14	5166650.6267	3324104.3060	1709450.1821	0.0074	0.0079	0.0078
BM13	5171918.9611	3316038.2355	1709102.7674	0.0054	0.0056	0.0041
BM10	5174347.8026	3307205.4067	1718751.3087	0.0046	0.0040	0.0047
BM8	5180095.0093	3303048.1454	1709472.2134	0.0069	0.0060	0.0063
TOEWR01	5180160.5494	3305084.7047	1705461.6146	0.0072	0.0059	0.0068
Kss6	5195258.9413	3296255.6144	1676529.2888	0.0070	0.0110	0.0134

Table.6: min and max standard deviation

Standard deviation	Min ± (m)	Max ± (m)	Mean ± (m)
σx	0.0046	0.0102	0.0079
σy	0.0040	0.0117	0.0081
σz	0.0041	0.0134	0.0084

Pre-analysis activities of the densified ground control stations adjustment covered such aspects as data screening, cycle-slip screening, outlier detection and ambiguity resolution. Furthermore, the normal equation files were created for every session. These normal equation files were combined in the final adjustment. In this manner several methods of fixing the geodetic datum were used and compared. The final combination gives the final co-ordinate file and the Variance–Covariance–Matrix of all co-ordinates (Table.5). In the post processing, the resulting repeatability can be indicated with better than 5mm in the east and north (horizontal precision), and better than 3cm in the height component (vertical precision) when referring to all stations (table.8) together with their root mean square (RMS) errors. Comparing table.3 (loop closure before adjustment) with table.10 (after the adjustment), it can be seen that, the loop Closure was properly made after the adjustment.

Table.7: Accuracy standard at 95% confidence level

station	E_{95} Of X(Cm)	E_{95} Of Y(Cm)	E_{95} Of Z(Cm)
BM7	1.9600	2.2932	2.4696
BM02	1.9992	1.9600	1.9404
BM01	1.7640	1.6856	1.7052
BM15	1.7836	1.9600	1.7444
BM16	1.9600	1.6660	1.8228
BM14	1.4504	1.5484	1.5288
BM13	1.0584	1.0976	0.8036
BM10	0.9016	0.7840	0.9212
BM8	1.3524	1.1760	1.2348
TOEWR01	1.4112	1.1564	1.3328
Kss6	1.3720	2.1560	2.6264

Table.8: Coordinates and horizontal precision and vertical precision and R.M.**S**

FROM	TO	Easting (m)	Northing (m)	Elevation (m)	H. Prec. ± (m)	V. Prec. ± (m)	R.M.S ± (m)
Kss4		454624.746	1716330.808	385.784			
	kss6	434959.582	1696037.291	398.045	0.003	0.013	0.018
	BM15	458707.352	1720766.463	383.704	0.003	0.019	0.01
	BM16	466804.928	1713629.715	386.294	0.004	0.018	0.017
	BM10	455523.796	1739798.173	381.411	0.003	0.019	0.019
	BM13	464250.835	1729763.457	387.355	0.004	0.019	0.02
	BM14	473880.828	1730104.672	409.956	0.003	0.018	0.019
	TOWER01	450581.576	1726003.646	408.989	0.005	0.024	0.013
	BM8	448909.735	1730177.818	381.669	0.005	0.025	0.031
	BM01	440575.873	1691144.514	384.461	0.003	0.016	0.018
	BM02	438284.612	1698683.644	383.957	0.003	0.015	0.017
	BM7	433249.986	1717675.2	389.468	0.003	0.015	0.019

In order to guarantee that all observations properly made, the quality assurance put on the consideration of short baseline, elevation mask and GDOP are unified. The observation was done in one session by using GNSS technology and the static carrier phase differential GNSS measurement then the data was processed by using Trimble Business Center to compute 3D coordinates for the network. The post processed data rigorously adjusted using least square method (observation equations) by developing computation software (MATLAB code). Eventually the data was accessed to the most probable value and accurate value for each point in the network, where a lot of analysis was made to guarantee the network quality, and the average errors in X, Y and Z were obtained to be 0.007,0.0081,0.0084 respectively

Khartoum State geodetic network is considered to be as a basis and as a development guide for typical governmental activities such as infrastructure, developing rural regions, building new settlements, and so forth. Moreover, building and maintaining this densification for the geodetic network can also be used as the foundations for georeferencing and Geographic Information Systems geodetic network base, which are considered to be as powerful tools for geospatial information and all kinds of governmental authority planning activities now and in the future. To meet the standards and specifications of spatial data, the survey team established the new geodetic control network to densify Khartoum State geodetic network by the processing and adjustment of the GNSS observations which, were carried out by Trimble Business Center software. The paper outlined the computation work carried out in this respect, in order to obtain the final set of co-ordinates and the network accuracy from the adjustment computation and constrained to the existing Khartoum ITRF2000 geodetic control stations (table.3). The resulting accuracy of the network, has been achieved to approximately 0.7cm in longitude and latitude, and 1cm in height. Several analyses of a different nature that were carried out confirmed the given accuracies.

Table.9: Station coordinates and horizontal precision and vertical precision

FROM	TO	$\sigma\Delta E$ ± (m)	$\sigma\Delta N$ ± (m)	$\sigma\Delta Ele$ ± (m)	$\sigma\Delta X$ ± (m)	$\sigma\Delta Y$ ± (m)	$\sigma\Delta Z$ ± (m)
kss7	kss6	0.001	0.001	0.005	0.004	0.003	0.002
	kss5	0.001	0.001	0.006	0.005	0.003	0.002
	kss4	0.001	0.001	0.007	0.006	0.004	0.002
	BM7	0.001	0.001	0.006	0.005	0.003	0.002
	BM02	0.001	0.001	0.006	0.005	0.003	0.002
	BM15	0.001	0.001	0.008	0.006	0.004	0.002
	BM16	0.001	0.001	0.006	0.005	0.003	0.002
	BM10	0.001	0.001	0.006	0.005	0.003	0.002
	BM13	0.001	0.001	0.006	0.005	0.003	0.002
	BM14	0.001	0.001	0.006	0.005	0.003	0.002
	TOWER01	0.002	0.002	0.008	0.007	0.004	0.003
	BM8	0.002	0.002	0.008	0.007	0.004	0.003
	BM01	0.001	0.001	0.006	0.005	0.003	0.002

Table.10: Sample of Typical loop Closure for these observing sessions after adjustment

$KSS4\ BM13\ BM10\ BM8\ KSS4$				
Baseline	Δx	Δy	Δz	Distance
$KSS4 - BM13$	-8222.2574	6142.9925	12961.3362	16532.9401
$BM13 - BM10$	2428.8396	-8832.8288	9648.5427	13304.60457
$BM10 - BM8$	5747.2101	-4157.2613	-9279.0936	11679.6757
$BM8 - KSS4$	46.2077	6847.0976	-13330.7853	14986.4845
Vector sum	0	0	0	56503.7048
Resultant closure= 0 ppm= 0				

In order to check for the internal quality and consistency of the data set, a loosely constrained adjustment of the densified network was firstly carried out (table.9). The quality of adjustment was also be independently verified at the checkpoint and the residuals are less than 1cm for the horizontal and 1.5cm for the vertical components, which indicates that the densified geodetic control network result is consistent with the existing Khartoum geodetic control network.

8. Conclusion

From the outlines and discussions given in the preceding sections, it can be concluded that, reliable geodetic reference network is a basic requirement for the successful execution of all survey and mapping related activities. The new 10 geodetic control points were established for the densification of Khartoum state, geodetic network. This new geodetic control points have been linked with the existing Khartoum geodetic network (ITRF2000.0). The paper indicated that, geodetic networks are generally appraised as a basis and as a development guide for government activities such as infrastructure, developing rural regions, building new settlements, and so forth. Moreover, building and maintaining geodetic network lays the foundations for Geographic, Land Information and geospatial systems, which are becoming, powerful tools for all kinds of governmental authority planning and decision-making activities now and in the future. The geodetic network established in is capable of providing a reliable basis for future developments as well as modern survey practice within the Khartoum area.

The Department of surveying Engineering team, first designed and established the GCPs Network in accordance with sound geodetic principles. Following a reconnaissance exercise, then built the required geodetic control points. The observation data of the GCPs campaign was collated and handed over to the processing team at Department geodetic laboratory. Subsequently, the survey team carried out the processing and adjustment of the GNSS observations. The paper highlighted the survey observations and computation work carried out in this respect. The resulting inner and outer accuracy of the network (local accuracy), has been achieved to approximately less than 1cm in the three-dimensional coordinates. This accuracy, is significantly better than that initially required by the survey team technical specifications. The resulting accuracy of the network, has been achieved to about 0.7cm in longitude and latitude, and 1cm in height component.

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