Aswan-Faculty of Engineering

Civil Engineering Department

ACCURACY ASSESSMENT STUDY OF USING GPS FOR SURVEYING APPLICAIONS IN SOUTH EGYPT

M.Sc. Thesis

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صلي ا علي وسلم

قال سبحانه وتعالي

بِسۡمِ ٱللَّهِ ٱلرَّحۡمَٰنِ ٱلرَّحِيمِ

وَلَوۡ أَنَّمَا فِي ٱلۡأَرۡضِ مِن شَجَرَةٍ أَقۡلَٰمٞ وَٱلۡبَحۡرُ يَمُدُّهُۥ مِنۢ بَعۡدِهِۦ
سَبۡعَةُ أَبۡحُرٖ مَّا نَفِدَتۡ كَلِمَٰتُ ٱللَّهِۗ إِنَّ ٱللَّهَ عَزِيزٌ حَكِيمٞ ٢٧

صدق الله العظيم

سورة لقمان

To my parents.

To my brothers and sister.

To my Wife.

To my family

To my friends.

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ABSTRACT

The Egyptian government announced for outstanding development plans for Upper Egypt area which will include many civil Engineering projects to get out from narrow Nile valley (5.3 % of total area) and use desert areas (94.7% of total area). GPS technology could be used effectively in this development plan. Thus accuracy assessment studies for using GPS technology in surveying applications in south Egypt is needed.

This research presents an accuracy assessment study of using different GPS observations techniques (static, stop and go and kinematic) for a number of surveying applications and assesses its behavior with classical surveying techniques where applicable.

During this research a series of field tests were executed to reflect characteristics of GPS collecting observations techniques.

The experimental work is divided to two main categories as follow;

- **Static GPS Technique**; this technique could be used for establishing surveying control points. During this category, the relative accuracy of points coordinates observed using GPS were assessed for a range of baseline lengths (2 - 31) km as well as for different observation times (10 - 120) minutes.
- **(Stop and Go) and Kinematic GPS techniques**; these techniques could be used for generation of Digital Elevation models as well as

generation of roads cross and longitudinal sections. During this category the use of GPS technology were assessed for two applications;

- Digital Elevation Model (DEM) generation :

A DEM was generated for (100 X 100) m^2 area using GPS (Stop&Go) technique as well as GPS kinematic technique. The accuracy of generated DEM using both GPS techniques was assessed with classical surveying measurements (total station).

- Road Longitudinal Section generation:

A 2km road longitudinal section was generated using Kinematic GPS Technique. The rover GPS receiver was set up over a moving vehicle during the collecting observation period. The GPS generated long. Section was assessed with classical surveying measurements (total station).

During this research; (L1 C/A code and carrier) GPS system ProMark 3.0 was used to collect GPS observations. GNSS Solution (3.00.07) software package was used to process all GPS observations. A Total Station TOPCON (GTS-712) was used for collecting classical surveying observations. Surfer 8.0 package was used to create all contour maps, grids and (cut and fill) volumes.

The findings of this research suggests that using GPS technology for surveying applications in south Egypt area is vital to safe of cost and time.

It can concluded that for Static GPS Technique, the relative accuracy for different baselines (95% Confidence) is 5 P.P.M. for every kilometer under conditions of a number of visible satellites not less than 6 and PDOP value not exceed 2.70.

It can concluded also that ; (Root Mean Square Error) RMSE for generated DEM using GPS (Stop and Go) technique comparing with classical survey observations is 9.736 cm, but for GPS Kinematic technique RMSE is 12.05 cm.

The Longitudinal section of road has a difference between 12.93 cm as Maximum value and -13.76 cm as Minimum value with RMSE of 5.99 cm. This accuracy is suitable for creating the contour maps and grids of the wide areas around the Lake Nasser to increase the green areas where the water is available to complete reclamation of these areas.

List of Symbols

Symbol	
ρ, ρt	the true distance to each satellite
ρ_S	the space coordinate of each satellite relative to the earth center
ρ_R	the ground receiver coordinate relative to the earth center
GPS	Global Positioning System
GNSS	Global Navigation Satellite System
GLONASS	GLObal NAvigation Satellite System
R	The true range for a satellite
δ	the receiver clock error "bias"
c	the velocity of light, approximately= 3×10^8 m/s
L1	First Carrier frequency
L2	Second Carrier frequency
λ	Wave length of satellite signal
f	the carrier frequency in Hz (cycles per second)
PRN	pseudo-random noise
C/A-code	Coarse/Acquisition-code
P-code	Precision-code
L5	The new Carrier frequency
Y- code	Military code
fo	the basic GPS frequency=10.23 MHz
S/A	Selective Availability
OCS	The Operational Control System
MCS	Master Control Station

Symbol	
AFB	Falcon Air Force Base
PPS	Precise Positioning Service
SPS	Standard Positioning Service
SEP	Spherical Error Probable
UTC	Universal Coordinate Time
A-S	Anti-Spoofing
NIS	Navigation Information Service
NGS	National Geodetic Survey
dt	the offset of the satellite clock from GPS time
dT	the offset of the receiver clock from GPS time
$d\rho_{ion}$	ionospheric error
$d\rho_{trop}$	tropospheric error
$\Delta(\rho)$	pseudo-range noise and multipath
φ	carrier phase measurements
$d\varphi_{ion}$	ionospheric error on carrier measurements
$d\rho_{trop}$	tropospheric error on carrier measurements,
N	integer ambiguity
$\Delta(\varphi)$	carrier phase measurements noise and multipath
N_e	the free electron density in the ionosphere (electron/m^3)
$A1$	a constant (40.3 if using S.I. units) (m^3/S^2)
TEC	Total Electron Content
ΔTrop	The tropospheric delay in m
SNR	signal-to noise ratio

Symbol	
DGPS	Differential Global Positioning System
OTF	on-the-fly
KNITs	Coordination Scientific Information Center
H	The Orthometric height
h	The ellipsoidal height
N1	Geoid undulation
MEO	Medium Earth Orbit
σ	standard deviation
ν	the residual of any measurement
M	any measurement
M'	the most probable value (mean)
n	numbers of measurements
RMSE	Root Mean Square Error
M_o	original value of measurement
$\sigma 1$	the position accuracy
σo	the measurement accuracy
D.O.P	Dilution of Precision
G.D.O.P.	Geometrical Dilution of Precision
P.D.O.P.	Position Dilution of Precision
H.D.O.P.	Horizontal Dilution of Precision
V.D.O.P.	Vertical Dilution of Precision
DEM	Digital Elevation Model
DTM	Digital Terrain Model
ppm	Part per million

Symbol	
UTM	Universal Transverse Mercator
δ'	Difference
δ'East	Difference in East coordinates.
δ'North	Difference in North coordinates
δ'H	Difference in Ellipsoidal heights
δ'L	Difference in Vector Length.
EGM	Earth Geopotential Model
WGS	World Geodetic System
S&G	Stop and Go kinematic GPS Technique
Δcf	The percentage difference in earthwork volume.
cf1	The net volume for total station survey.
cf2	The net volume for the others techniques.

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Chapter (1)
Introduction

CHAPTER (1)

1.1 INTRODUCTION

The Global Positioning System (GPS) is the only fully functional Global Navigation Satellite System (GNSS), GPS is used to determine the position, velocity and time thus GPS Technology has different applications in different fields such as Industry, Agriculture, Mapping, Geographical Information System (GIS) Data Collection, Public Safety, Surveying, Telecommunications, Military (Intelligence & Target Location, Navigation and Weapon Aiming & Guidance), Science (Archeology, Atmospheric sciences, Environmental, Geodesy, Geology & Geophysics, Oceanography and Wildlife) and Transportation.

GPS technology proved its strength in surveying Engineering applications especially in developing countries such as Egypt through a number of years. Yet the dependence on GPS technology as a surveying tool is poor in South Egypt (Upper Egypt). South Egypt represents nearly half the area of Egypt with minor percentage of population.

The Egyptian government announced for outstanding development plans for Upper Egypt area which will include many civil Engineering projects. GPS technology could be used effectively in this development plan. Thus accuracy assessment studies for using GPS technology in surveying applications in south Egypt is needed.

1.2 Thesis Objectives

This thesis is an example of such effort where it represents accuracy assessment field study for GPS techniques (Static, (Stop and Go) and kinematic) in Engineering GPS applications such as positioning accuracy and DEM generation. The experimental tests were carried out in Aswan Government.

During field studies, ProMark 3.0 GPS system and Total Station Topcon GTS-712 were used, GNSS Solution 3.00.07 software package was used to process all GPS data, and Surfer 8.0 was used to create all contour maps, grids and to calculate cut and fill volumes.

1.2.1. Static GPS technique

Static GPS surveying is a relative positioning technique (see 2.8.2) that depends on the carrier-phase measurements. It employs two (or more) stationary receivers simultaneously tracking the same satellites (see Figure 2-16). One receiver, the base receiver, is set up over a point with precisely known coordinates. The other receiver, the remote receiver, is set up over a point whose coordinates are desired. This type is important in applications of creating control points in local traverses and geodetic networks.

This part had two sets,

- The first set was surveying different length base-lines ranges (0 to 20) km, and this part was surveyed on Aswan –Edfo Desert Road.
- The second set was surveying a 31 km baseline from Aswan Faculty of Engineering to Daraw city in Aswan Governorate.

1.2.2. Kinematic GPS technique

Kinematic GPS surveying is the most productive to have the greatest number of points in least time .The drawback of this technique is the continuous lock of at least four satellites. The coordinates of the surveying points obtained after finishing the surveying in the field as a post-processing. Kinematic GPS technique consists of two branches of surveying

1.1.2.1. Stop-and-Go GPS surveying (Semi-kinematic)

Stop and Go is characterized by stopping and moving one receiver to determine the positions of fixed points along the trajectory. In this technique the relative positional accuracies are at centimeter level. This type is important to study the possibility of using GPS in creating grids and contour maps for leveling applications for new construction projects and reclamation of desert. The new direction of Egyptian Government is to increase the populated and green areas and to get out from narrow Nile valley to wide desert areas; especially in Aswan, where there are wide areas a round Lack Nasser behind High Dam

1.1.2.2. kinematic GPS surveying

This technique is best for wide open areas depending on using base point and rover receiver (surveyor or rover car).

This part of experimental work was surveyed in the new city of EL-SADAKA - Aswan governorate and this work includes two sets,

- First is creating the Digital Elevation Model (DEM) using Stop and Go technique, Kinematic, and Total Station (reference data). The aim of this set is to find the accuracy of using GPS techniques in height compared with Total Station height to create DEM.

- The second experimental a road is surveyed by Total Station and was used as a reference. The same road surveyed by a moving car with Kinematic GPS technique. A car with speed in between 15 to 20 km/hour was used. This type is used in road planning applications.

1.3 Thesis Main Findings

- For Static GPS Technique; it can be concluded that the base line relative accuracy is 5 ppm (5 mm per 1 km).
- For DEM; The residuals values for Stop and Go technique were in range between 3.1 cm to 11.6 cm and for kinematic technique were in range between (-14.5 cm to 25.5) cm.
- For Road Longitudinal section by Kinematic technique; the difference between the Total Station technique and Kinematic (by moving car) was in between -13.7 to 12.9) cm with RMSE of 6.0 cm.

1.4 Thesis Organization

This thesis is organized as follows

- **Chapter 1**: this chapter contains an introduction and lists the objectives of the research.
- **Chapter 2**: this chapter contains the literature review about Global Navigation Satellite System Technology (over view, components and applications).
- **Chapter 3**: this chapter describes the accuracy assessment and methods of doing it.

- **Chapter 4**: this chapter contains the experimental work of the thesis and the analysis of the results.
- **Chapter 5**: this chapter contains the conclusions of this investigation.

Chapter 2

GNSS Technology Overview

GNSS Technology Overview

2.1 Introduction

This chapter aims to give the reader a short overview about Global Navigation Satellite System (GNSS) technology characteristics. GNSS systems are; the American GPS system, the Russian GLONASS system, the European Galileo system. GPS system is the only fully functional Global Navigation Satellite System (GNSS), GLONASS system is not yet fully functional global system as the number of constellation satellites is less than the designed number of satellite (24) and Galileo is the future European GNSS system under civilian control which is planned to be fully functioning around 2012, so this chapter contains of the following items:

- Basic Concepts of GNSS technology.
- GPS System Overview.
- Positioning Services offered by GPS.
- Navigation message and Satellite Ephemeris.
- GPS Observations.
- GPS Errors.
- GPS Positioning Techniques.
- GLONASS system overview.
- Galileo system overview
- Datum and Map projection of GPS Surveying; preview of horizontal, vertical geodetic datum and map projections.
- Digital Elevation Model preview.

2.2 Basic Concepts

GPS technology is used for the instantaneous determination of position, velocity and time, GPS is a ranging system from known positions of satellite in space to unknown positions on land, sea, air and space, GPS provide the user the capability to determine his position "latitude, longitude and elevation" by measure distances from a number of satellites to the user position.

Assume that the satellite frozen at instant, the space coordinate of each satellite relative to the earth center (ρ_S) can be computed from the ephemeris broadcast by the satellite, if the ground receiver by geocentric position vector (ρ_R), the true distance (ρ) to each satellite could be accurately measured by recording the time required for the satellite signal to reach the receiver ,by using this technique we need intersection of three spheres for three unknowns " latitude, longitude and elevation " which could be determined from three range equations

$$\rho = || \rho_S - \rho_R || \qquad (2\text{-}1)$$

ρ is the true distance to each satellite.

ρ_S *is* the space coordinate of each satellite relative to the earth center.

ρ_R is the ground receiver coordinate relative to the earth center.

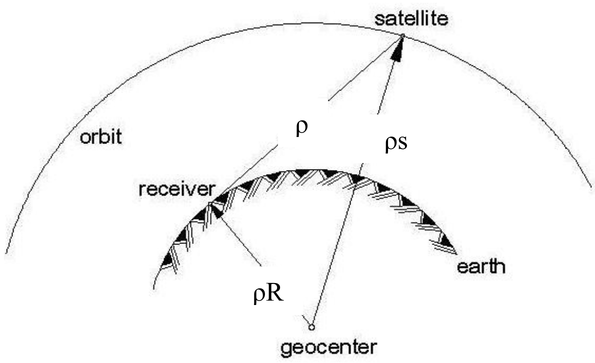


Fig. (2-1)Principle of satellite positioning.

[Hoffmann-Wellenhof (2000)]

The time clock of satellite is atomic crystal clock that measure the time of the transmitted signal from satellite in GPS time, but the time clock of ground receiver has an offset from true GPS time because it isn't an accurate as satellite clock, so we must add a range correction from receiver clock error, the pseudo-range R is equal the true range plus the range correction ($\Delta\rho$) resulting from the receiver clock error "bias" δ

$$R= \rho+ \Delta\rho=\rho+ c\ \delta \qquad (2\text{-}2)$$

Where: c is the velocity of light.

So we need four measurements to solve the four unknowns, three components of position and the clock bias. [Hoffmann-Wellenhof (2000)]

2.3 System Overview

The Global Positioning System (GPS) consists of a constellation of radio-navigation satellites (Space Segment), a ground control segment to manage satellite operation and user segment with receivers who uses the satellite data to satisfy a broad range of positioning requirements, (see Figure 2-2) .[Natural Resources Canada (1995)]

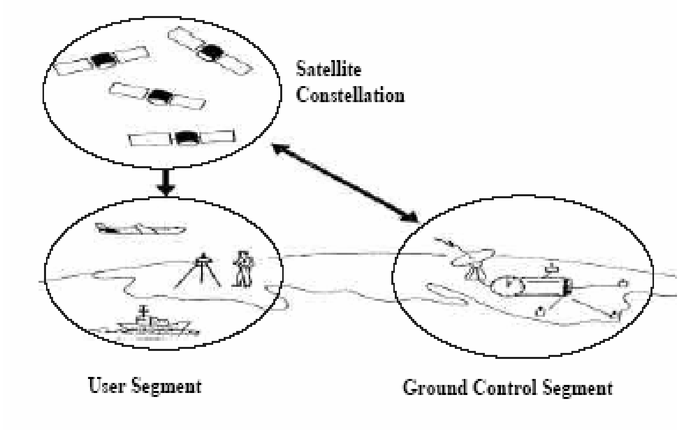


Fig. (2-2)GPS Components.

[Natural Resources Canada (1995)]

2.3.1 Space Segment

2.3.1.1 Satellite Constellation:-

The GPS satellites have nearly circular orbits with an altitude of about 20200 km above the earth; the designed constellation consists of 24 operational satellites in six spaced planes (A to F) with an inclination of 55º and with four satellites per plane and a spare satellite; with this constellation the GPS provide 24-hour world wide coverage (see Figure 2.3). [Hoffmann-Wellenhof (2000), El-Rabbany Ahmed (2002)]

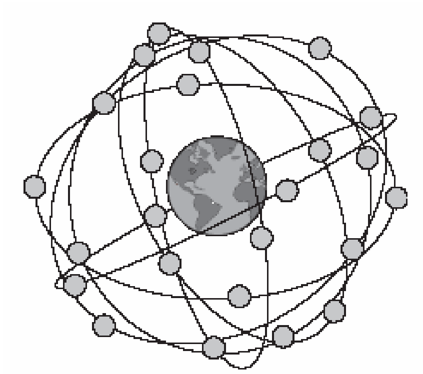


Fig. (2.3)GPS Constellation.

[El-Rabbany Ahmed (2002)]

Since early days of year 2009 up till now, the numbers of the tracking satellites are 31 satellites as shown in Table (2-1)

Table (2-1): Satellite Numbers.

[NAVCEN (2009) a]

Orbital Plan	SVN
A	6 satellites
B	5 satellites
C	5 satellites
D	5 satellites
E	5 satellites
F	5 satellites

2.3.1.2. Satellite Signals

Signals from GPS satellites are continuously transmitting two *carrier frequencies*, L1=1575.42 MHz and L2=1227.60 MHz, since radio waves propagate through space at the speed of light, the wavelengths of the GPS carrier signals are computed as

$$\lambda = c / f \qquad (2.3)$$

Where $\underline{\lambda}$ is the wavelength (the length of one cycle) in meters, $\underline{c}$ is the speed of light (approximately= 3×10^8 m/s) and $\underline{f}$ is the carrier frequency in Hz (cycles per second) (see Figure 2-4). [Natural Resources Canada (1995)]

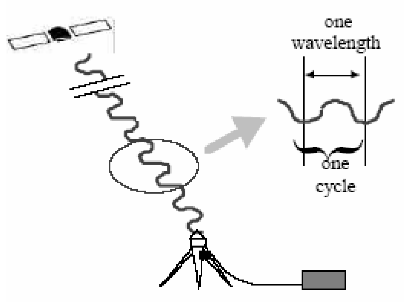


Fig. (2-4) Satellite signal Carrier.

[Natural Resources Canada (1995)]

These dual frequencies are essential to eliminate the major source of error, (ionospheric refraction). The pseudo-ranges that are computed from measured travel times of the signal from each satellite to the receiver use two pseudo-random noise (PRN) codes that are modulated upon the two base carrier waves.

- The first code is the C/A-code (Coarse/Acquisition-code), also designated as the Standard Positioning Service (SPS), which is available for civilian use. The C/A-code with an effective wavelength of 300 m is modulated only on *L1* and is purposely omitted from *L2*.

- The second code is the P-code (Precision-code), also designated as the Precise Positioning Service (PPS), which has been reserved for the U.S. military and other authorized users. The P-code has an effective wavelength of 30 m is modulated on both carriers L1 and L2 (see Figure 2-5). [Hoffmann-Wellenhof (2000), EUROCONTROL (1998)]

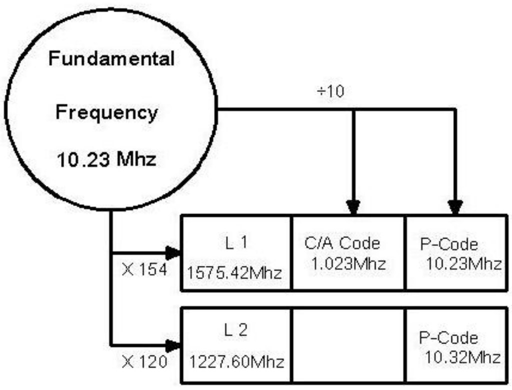


Fig. (2-5)GPS Signals Flow chart.

- There is a signal modernization of GPS satellites after the first generation of satellites; the next generation of satellites has many improvements over the first satellites to include the capability to transmit data between satellites to make the system more independent, to give the satellites the capability to Autonomous Navigation (Auto-Nav) to allow the satellites to position themselves without ground control. According to the signal modernization in 1999, future GPS signals will be transmitted by three civil carriers L1, L2, L5 as follow

$$\left.\begin{aligned} L1 &= 154\,fo = 1575.42 \text{ MHz} \\ L2 &= 120\,fo = 1227.60 \text{ MHz} \\ L5 &= 115\,fo = 1176.45 \text{ MHz} \end{aligned}\right\} \quad (2.4)$$

Where fo = 10.23 MHz denotes the basic GPS frequency.

The carriers L1 and L2 will be modulated with C/A code but L5 will be modulated with a new civil code similar to the P-code. The existing military Y- code will be replaced by new split M-code [Hoffmann-Wellenhof (2000)]. L5, the third carrier will be available in the Block IIF satellites [NAVCEN (2009) b], see Figure (2-6).

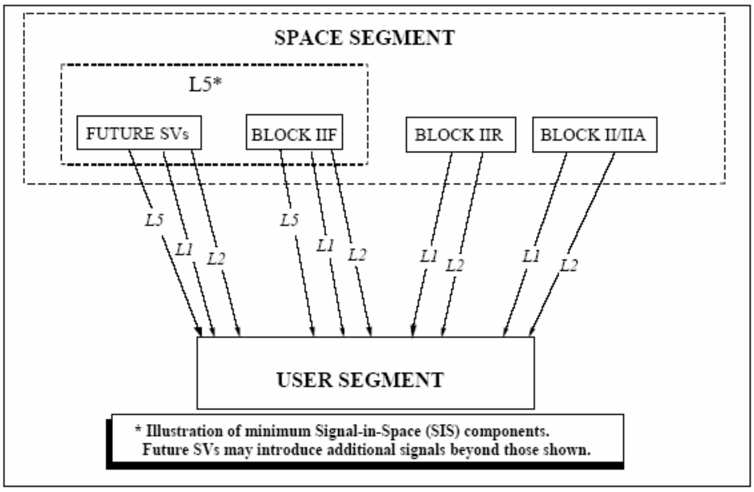


Fig. (2-6) GPS Space Segment to User Segment Interfaces.

[NAVCEN (2009) b]

2.3.1.3. System Denying Accuracy Techniques

There are basically two methods for denying civilian users full use of the GPS system.

- Selective Availability (S/A)

The goal of SA was to reduce the accuracy of GPS positioning, velocity and time to introduce pseudo-random errors into the satellite signals. SA has been in force since March 1990 according to the specifications of DoD, the accuracy of stand alone receivers was degraded to 100 m for horizontal position and 156 m for height at 95% accuracy confidence .SA was turned off (reduced to zero) on May, 2000 then the accuracy of stand alone receivers improved by a factor of 10. [Hoffmann-Wellenhof (2000)]

- Anti-Spoofing (A-S)

The A-S feature is activated on all satellites, This feature encrypts the P-code into the Y-code .Encrypting keys are available to PPS users only, which allow them to remove the effects of A-S and obtain maximum accuracy of GPS. [El-Rabbany Ahmed (2002)]

2.3.2 Control Segment

The Operational Control System (OCS) consists of a master control station, monitor stations and ground control stations. The main operation of the OCS are tracking the satellites for the orbits and clock determination and prediction, time synchronization of the satellites, and upload the data message to the satellite.

- Master Control Station (MCS)

At Falcon Air Force Base (AFB) in Colorado Springs, USA, MCS collects the tracking data from the monitor stations and calculate the satellite orbit and clock parameters using a Kalman filter.

- Monitor Stations

There are five monitor stations located at: Hawaii, Colorado Springs, Ascension Island in South Atlantic Ocean, Diego Garcia in Indian Ocean, and Kwajalein in the North Pacific Ocean. Each station is equipped with a precise atomic time standard receivers which continuously measure pseudo-oranges to all satellite in view. Pseudo-ranges are measured every 1.5 seconds and using the ionospheric and meteorological data, they are smoothed to produce 15 minutes interval data which are transmitted to the master control station. [Hoffmann-Wellenhof (2000)]

- Ground Control Stations

These stations at Ascension, Diego Garcia and Kwajalein, these stations are used to periodically upload the ephemeris and clock data to each satellite for re-transmission in the navigation message.

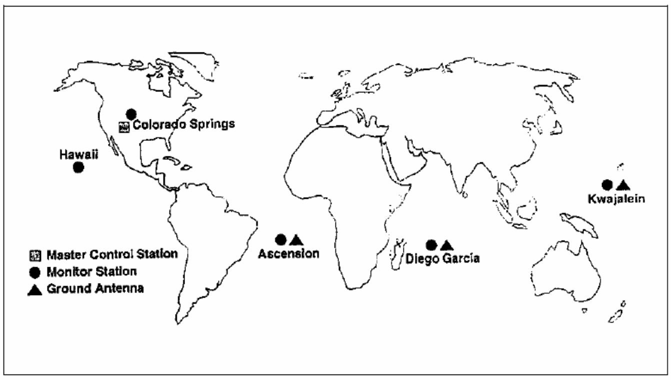


Fig. (2-7)GPS Stations.

[NAVSTAR (1996)]

2.3.3 User Segment

The user segment consists of receivers designed to receive, decode, and process the satellite signals. The user segment represents the ground-based receiver units that process the GNSS satellite signals and arrive at a position of the user. It consists of both military and civil activities for an almost unlimited number of applications in a variety of air, sea, and land. [NAVSTAR (1996)]

2.4 GPS Positioning Services:-

2.4.1 Precise Positioning Service(PPS)

The Precise Positioning Service is an accurate positioning, velocity and timing service but is available to military purposes .The PPS uses the P-code on both frequencies L1 and L2, and the C/A code on L1 frequency, the PPS is provide 16 m Spherical Error Probable (SEP) (3-D, 50%) positioning accuracy and 100 nanosecond Universal Coordinate Time (UTC) time transfer accuracy ,this is approximately equal to 37 m (3-D, 95%).The high accurate of PPS founded by the two controlling features S/A and A-S which S/A was reduced to zero on May 2000 and the A-S encryption code is provided to the PPS users. [Farah Ashraf (2004)]

2.4.2. Standard Positioning Service(SPS)

The SPS is available to all GPS users but it has a less accurate positioning and timing service. The SPS is available to civilian users that use C/A code on L1 frequency only, the level of SA was controlled to provide 100 m (95%) horizontal accuracy which approximately equal to 156 m (3-D, 95%) and 337 nanosecond Universal Coordinate Time (UTC) time transfer accuracy. The performance of the SPS service improved after SA was reduced to zero, giving a global average positioning accuracy of 33 m 95% horizontal error and 73 m 95% vertical error with time transfer accuracy of 40 nanoseconds 95%. [GPS-SPS (2001)]

2.5 Navigation message and Satellite Ephemeris

2.5.1 Navigation message

The navigation message contains information about the satellite health status, the satellite clock, the satellite orbit, and various correction data. The total message containing 1500 bits is subdivided into five subframe .One subframe is transmitted in 6 seconds and contains 10 words with 30 bits. The transmission time needed for a word 0.6 seconds. Note that a receiver requires at least 30 seconds locking on a satellite in order to receive the complete navigation message (see Table 2-2).

Table (2-2) Scheme of navigation message.

[Hoffmann-Wellenhof (2000)]

	Number of bits	Transmission time
Total message	1500	30 seconds
Sub frame(1-5)	300	6 seconds
Word (1-10)	30	0.6 seconds

The first sub frame contains the GPS week number, a prediction of the user range accuracy, indicators of the satellite health and of the age of the data, an estimation of the signal group delay, and three coefficients for a quadratic polynomial to model the satellite clock correction.

The second and third subframe transmits the broadcast ephemeris of the satellite.

The fourth and the fifth subframe contents are changed in every message, the fourth subframe are reserved for military use; the remaining contain information on the ionosphere, UTC data, the almanac data (low accuracy orbital data) for satellites beyond the nominal 24 constellation. The fifth subframe is mainly dedicated to the almanac data and the health status for the first 24 satellites in orbit. [Hoffmann-Wellenhof (2000)]

2.5.2. Satellite Ephemeris

There are three main types of available data to determine position and velocity vectors of the satellites in a terrestrial reference frame at any instant: almanac data, broadcast ephemeris and IGS ephemeris. Almanac and broadcast ephemeris are broadcasts from the GPS US government agencies but the IGS is an international agency provides many types of ephemeris (Broadcast, Rapid, Ultra-Rapid and final) [IGS (2009)]. The data are different in accuracy and there are available in real time or after the fact. (See Table 2-3)

Table (2-3) Uncertainties of Ephemeris.

[Hoffmann-Wellenhof (2000)]

Ephemeris	Uncertainty	Remark
Almanac	Some kilometers	Depending on the age of data
Broadcast	1 m	Or even better
Final IGS	0.05-0.20 m	Depending on the delay

2.5.3. Almanac data

The almanac data provide the user with *less precise* data to facilitate receiver satellite search or for planning tasks such as the computation of visibility charts. The almanac data are uploaded at least every 6 days and are broadcast as part of *the satellite navigation message*, the almanac message contains parameters for the orbit and satellite clock correction terms for all satellites as shown in (see Table 2-4). Almanac data are available from a variety of information service such as Navigation Information Service (NIS), (*www.navcen.uscg.mil*); the data are packed into the YUMA or SEM formats.

Table (2-4): Almanac data.

[Hoffmann-Wellenhof (2000)]

Parameter	Explanation
ID	Satellite PRN number
WEEK	Current GPS week
ta	Reference epoch in seconds within the current week
$\sqrt{a}$	Square root of semi major axis in meter
e	Eccentricity
Mo	Mean anomaly of reference epoch
ω	Argument of perigee
δi	Inclination offset from 0.3 semicircles (≈54°)
ιο	Longitude of the node's at weekly epoch
$\dot{\Omega}$	Drift of node's right ascension per second
ao	Satellite clock offset in seconds
a1	Satellite clock drift

2.5.4. Broadcast Ephemeris

The broadcast ephemeris based on observations at the monitor stations of the GPS control segment .The recent of these data used to compute a reference orbit for the satellites .Additional tracking data are entered into a Kalman filter to improved the satellite orbits .The Master Control Station is responsible for the computation of the ephemeris and upload it to the satellites.

The broadcast ephemeris are a part of *the satellite navigation message*; these ephemeris contain records with general information, records with orbital information, and records with information of the satellite clock as shown in (see Table 2-5).

Table (2-5): Broadcast ephemeris.

[Hoffmann-Wellenhof (2000)]

Parameter	Explanation	Parameter	Explanation
ID	Satellite PRN number	*Δn*	Mean motion difference
WEEk	Current GPS week	*Ω̇*	Rate of node's right ascension
Te	Ephemeris Reference epoch	*Cuc, Cus*	Correction coefficients(argument of perigee)
√a	Square root of semi major axis in √meter	*Crc, Crs*	Correction coefficients(geocentric distance)
E	Eccentricity	*Cic, Cis*	Correction coefficients(inclination)
Mo	Mean anomaly of reference epoch	*Tc*	Satellite clock reference epoch
Ωo	Argument of perigee	*Ao*	Satellite clock offset
Io	Inclination	*a1*	Satellite clock drift
Io	Longitude of the node's at weekly epoch	*a2*	Satellite clock frequency drift

The ephemeris is broadcast mostly every hour and should only be used during the prescribed period of approximately *four hours*.

2.5.5. Final IGS Ephemeris

The IGS ephemeris consists of satellite positions and velocities at equidistant epochs. The most accurate orbital information is provided by the IGS with a delay of about two weeks (Final IGS ephemeris) but IGS provides many types of ephemeris as shown in Table (2-6)

Table (2-6): IGS Products.

[IGS (2009)]

GPS Ephemeris		Accuracy	Latency	Updates	Sample Interval
Broadcast	orbits	~160 cm	real time	--	daily
	Sat. clocks	~7 ns			
Ultra-Rapid (predicted half)	orbits	~10 cm	real time	four times daily	15 min
	Sat. clocks	~5 ns			
Ultra-Rapid (observed half)	orbits	<5 cm	3 hours	four times daily	15 min
	Sat. clocks	~0.2 ns			
Rapid	orbits	<5 cm	17 hours	daily	15 min
	Sat. & Stn. clocks	0.1 ns			5 min
Final	orbits	<5 cm	~13 days	weekly	15 min
	Sat. & Stn. clocks	0.1 ns			5 min

Note 1: IGS accuracy limits, except for predicted orbits, based on comparisons with independent laser ranging results.

Note 2: The accuracy of all clocks is expressed relative to the IGS time scale, which is linearly aligned to GPS time in one-day segments. [IGS (2009)]

2.6 GPS Observations

There are two types of GPS observations: (i) *Code pseudo-range*, which used for navigation and (ii) *Carrier phase pseudo-range*, used for high precision positioning applications as shown in Fig (2-8)

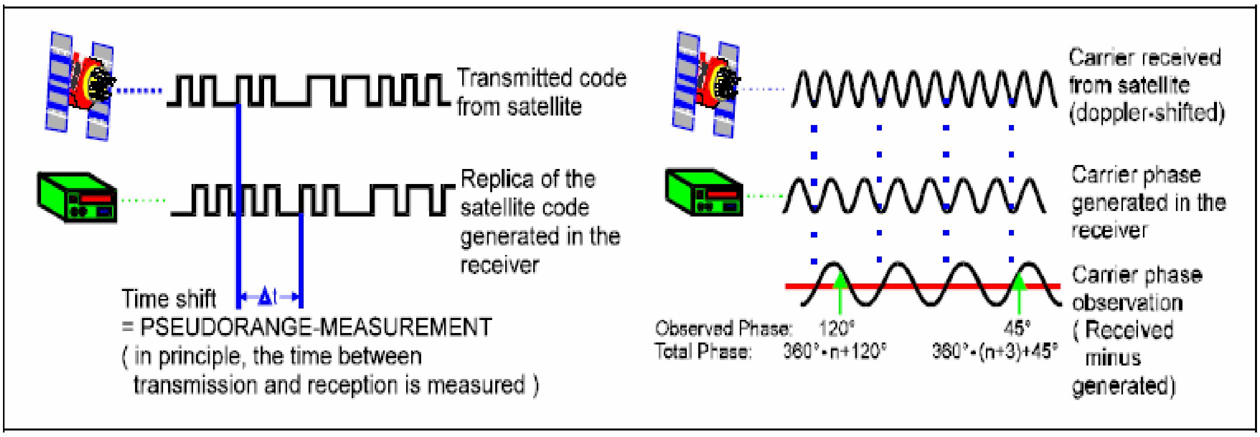


Fig. (2-8) GPS Observables.

EUROCONTROL (1998)

2.6.1 Code Pseudo-range

The time difference between the transmitted and received GPS signal, obtained via a code correlation process, multiplied by the speed of light in a vacuum to give the pseudo-range between the satellite and the receiver .Also there are some errors in the synchronization of the satellite and receiver clocks with each other and with GPS time, the resulting distance is called a "pseudo-range", not the true range as given:

$$\rho = \rho t + c\,(dT - dt) + d\rho_{ion} + d\rho_{trop} + \Delta(\rho) \qquad (2\text{-}5)$$

where,

ρ : observed pseudo-range measurement,

ρt : true range,

c : speed of light in a vacuum,

dt : the offset of the satellite clock from GPS time,

dT : the offset of the receiver clock from GPS time,

$d\rho_{ion}$:ionospheric error,

$d\rho_{trop}$:tropospheric error,

$\Delta(\rho)$:pseudo-range noise and multipath.

The true range is a function of the receiver and the satellite positions. The satellite positions are obtain using the satellite ephemeris and the satellite clock correction parameters which are given in the broadcast navigation message .when the ionospheric and tropospheric errors are modeled ,the only unknowns are the receiver clock bias and the coordinates of the receiver position ,so the observed pseudo-range measurement expressed as:

$$\rho = \rho t + c\,(dT - dt) + \Delta(\rho) \qquad (2\text{-}6)$$

with $\rho t = \sqrt{(Xs - Xr)^2 + (Ys - Yr)^2 + (Zs - Zr)^2}$

where

(Xs,Ys,Zs) : satellite coordinates,

(Xr,Yr,Zr) : receiver coordinates.

The un-modeled errors such as multipath and measurement noise are assumed to be random and are included as a residual term in a least squares solution.

2.6.2. Carrier Phase Pseudo-range

The carrier phase observable obtained by removing the code from the incoming signal. The approximate wavelengths of the carrier frequencies are 19 cm for L1 and 24 cm for L2. The phase difference of the carriers can be accurately measured to be better than 0.01 cycles so it is possible to obtain very precise positioning using the carrier phase observable. The distance between the satellite and the receiver is obtained by measuring the fractional part of the carrier phase, plus the total number of cycles between the satellite and the receiver as shown:

$$\lambda\varphi = \rho_t + c\,(dT - dt) - \lambda d\varphi_{ion} + \lambda d\rho_{trop} - \lambda N + \Delta(\varphi) \qquad (2\text{-}7)$$

where

φ	:carrier phase measurements,
$d\varphi_{ion}$	:ionospheric error on carrier measurements,
$d\rho_{trop}$	:tropospheric error on carrier measurements,
λ	:carrier wavelength,
N	: integer ambiguity,
$\Delta(\varphi)$	: carrier phase measurements noise and multipath.

The receiver is able to count the change in the number of cycles but not the number of cycles traveled through before the receiver starts to count the cycles, which referred to as the 'integer ambiguity'. The integer ambiguity remains the same as long as the receiver phase lock up the satellite but if the signal is lost by 'cycle slip', the 'integer ambiguity needs to be computed again (see Figure 2- 9,2-10) . [Farah Ashraf (2004)]

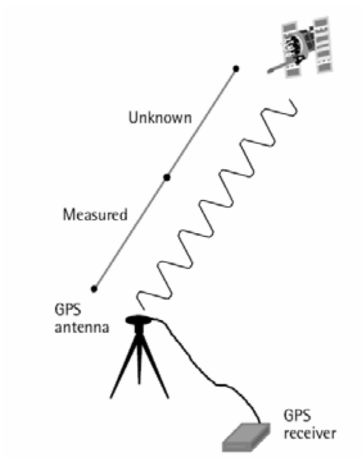


Fig. (2-9) Integer Ambiguity.
El-Rabbany Ahmed (2002)

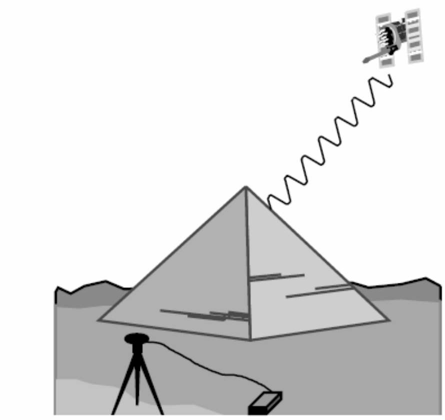


Fig. (2-10) Cycle Slip.
El-Rabbany Ahmed (2002)

The values of errors in the two observation modes as shown in the following table (see Table 2-7).

Table (2-7): Errors of observation modes.
[EUROCONTROL (1998)]

C/A code Observations	Receiver Noise	Differential Position Error (PDOP=3)
Pseudo-range	0.2 …..5.0 m	0.6 …..15 m
Carrier Phase	0.2 …..2.0mm	0.6 …..6 mm

2.7 GPS Errors

GPS measurements are subject to three main types of errors; (i) gross errors, (ii) systematic errors (iii) random errors. Gross errors are outliers in the observation model. The systematic errors must remove from the observations to avoid adding biases to the results. These errors have some physical or mathematical relationship with the measurements and can be modeled as additional terms in the observation equation, or eliminated by combinations of the observations. The random errors are the discrepancies remaining after the systematic errors and gross errors have been removed. The systematic errors can be described as follow according to their source. [Farah Ashraf (2004)]

2.7.1 Satellite Dependent Errors

2.7.1.1 Satellite Orbital Error

The satellite orbit information provided through the broadcast ephemeris in the navigation message. The positions of the satellites are treated as known parameters during GPS data processing to solve for the unknown receiver position. Therefore any error in the satellite positions propagates directly into the calculated receiver coordinates. The accuracy of the satellite position given in the broadcast ephemeris is in the range of (5 – 10) meters [David C. Jefferson and Yoaz E. Bar-Sever (2000)]. The IGS (International GPS Service) offers different types of GPS ephemeris with different accuracies. The accuracy of the GPS broadcast ephemeris equals to around 160 cm daily and the final IGS ephemeris is estimated less than 5 cm but is only available after a 13 days delay. [IGS (2009)]

2.7.1.2 Satellite Clock Error

GPS satellites carry both rubidium and cesium atomic frequency standards. The clocks are left to drift off the GPS time but their drift is monitored by the control segment. The clock error is included in the broadcast navigation message [Farah Ashraf (2004)].The size of the satellite clock bias without corrections (broadcast ephemeris) is of the order of 7 ns, but in final precise ephemeris it reduced to less than 0.1 ns. [IGS (2009)]

2.7.1.3 Relativistic Effects

The special relativity is due to the satellite's velocity, whereas the general relativity is due to the drift in the gravitational potential at the satellite's position relative to the gravitational potential at the Earth's surface.

2.7.2 Receiver Dependent Errors

2.7.2.1 Receiver Clock Error

Most GPS receiver clocks are based on quartz oscillators. The receiver clock error which biases measurements with respect to GPS time, this error removed when observations are differenced with respect to two satellites.

2.7.2.2 Antenna Phase Center Variations

The phase center of an antenna is the electrical point to which the GPS satellite signal is referred and generally is not identical to the geometric center of the antenna. The offset is dependent on the elevation, azimuth and intensity of the satellite signal and is different for the L1 and the L2 observations. [Hoffmann-Wellenhof (2000)]

2.7.3 Signal Path Dependent Errors

GPS measurements are subject to errors when the signal passes through the earth's atmosphere which bends the ray and also slows it. The excess of signal path length are due to two components propagation delay and signal bending [Donson. A. H. (1986)], but in most geodetic purposes these errors can be modeled. The atmosphere can be considered as two distinct components, the ionosphere and the troposphere (see Figure 2-11), which have different physical characteristics and should be modeled separately. [Farah Ashraf (2004)]

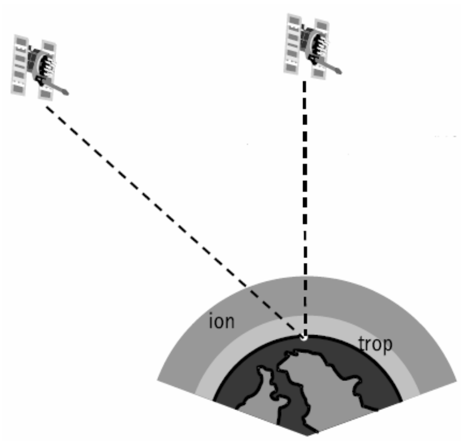


Fig. (2-11) Atmosphere components.

El-Rabbany Ahmed (2002)

2.7.3.1 Ionospheric Errors

The ionosphere layer extends from about 50 km to 1100 km above the earth [Seeber, G. (1993)]. The relationship between the refractive index *n*, of the ionosphere and the frequency is given by the following equation. [Donson. A. H. (1986)]

$$n = 1 \pm \frac{A1Ne}{f^2} + \text{higher order terms} \qquad (2\text{-}8)$$

Where,

$A1$ is a constant (40.3 if using S.I. units) (m^3/s^2)

N_e is the free electron density in the ionosphere (electron/m^3),

f is the frequency of the signal (Hz).

The sign ± in the previous equation taken (+) for the code range observation that the observed range is too long, but taken (–) for the carrier phase range that that the observed range is too short. The ionospheric delay errors vary from 10 cm to 10 m depending on the Total Electron Content (TEC) along the signal path through the ionosphere. Since the ionospheric delay is dependent on the signal frequency, the delay can be estimated through the combination of dual frequency observations. [Farah Ashraf(2004)]

2.7.3.2 Tropospheric Errors

The troposphere is the electrically neutral atmospheric region that extends up to about 50 km from the surface of the earth .The troposphere is a non-dispersive medium for radio frequencies below 15 GHz [El-Rabbany Ahmed (2002)]. As a result, it delays the GPS carriers and codes identically. That is, the measured satellite-to-receiver range will be longer than the actual geometric range, which means that a distance between two receivers will be longer than the actual distance. Unlike the ionospheric delay, the tropospheric delay cannot be removed by combining the L1 and the L2 observations. This is mainly because the tropospheric delay is frequency independent.

Tropospheric delay may be broken into two components, dry and wet. The dry component represents about 90% of the delay and can be predicted to a high degree of accuracy using mathematical models. The wet component of the tropospheric delay depends on the water vapor along the GPS signal path. Unlike the dry component, the wet component is not easy to predict [Leick (1995)].The tropospheric delay ΔTrop (meters) is equivalent to the integral of the refractive index along the tropospheric signal path, which can be given as,

$$\Delta \text{ Trop} = 10^{-6} * \int_a^b N ds \qquad (2\text{-}9)$$

Where

$N = (n\text{-}1) \times 10^6$, is the refractivity,

a and *b* define the limits of the troposphere boundary.

2.7.3.3 Cycle Slips and Multi-path

- Cycle Slips

A cycle slip is caused by the loss of lock in the phase-lock loop, generating a discontinuity in the accumulation of the integer number of cycles (N) [El-Rabbany Ahmed (2002)]. This phase ambiguity integer number (N) remains constant as long as no loss of the signal lock occurs .In this event the integer counter is reinitialized which causes a jump is called *cycle slip* (see Figure 2-12). [Hoffmann-Wellenhof (2000)]

Loss of lock may occur for three reasons: First, obstruction of the satellite signal due to trees, building, and bridges. The second source is a low signal-to noise ratio (SNR) due to atmospheric disruption, multipath, high receiver dynamics and low satellite elevation. The third source is receiver software failure. The cycle slips have to be detected and repaired to avoid any bias in the measurements.

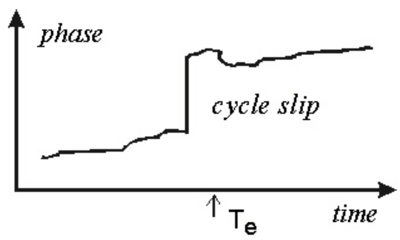


Fig. (2-12) Drop of phase due to Cycle Slip.

[Hoffmann-Wellenhof (2000)]

- Multipath

The multipath effect is caused by the arrival of the signal at the receiver via more than one path due to the presence of reflecting surfaces near the receiver (see Figure 2-13). Multipath can also occur due to reflections at the satellite itself during signal transmission. While both code and carrier measurements are affected by multipath, the effect on code is two orders of magnitude larger than on carrier phase observations. [Seeber, G. (1993)]

As multipath effects depend on the surrounding environment of the receiver, there is no general model to correct for these effects. However, when the same environmental conditions exist, repetitive patterns can be found in many cases from day to day static observations. Possible ways to minimize multipath effects are: carefully chosen receiver sites, carefully designed antennas and accessories such as ground planes and choke rings (see Figure 2-14). [Farah Ashraf (2004)]

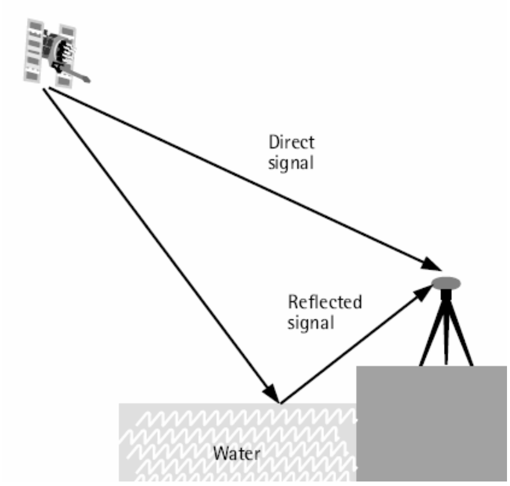


Fig. (2-13) Multipath effect.

El-Rabbany Ahmed (2002)

Fig. (2-14) Choke Ring.

El-Rabbany Ahmed (2002)

2.8 GPS Positioning Techniques

Positioning with GPS can be performed by either of two ways: point positioning or relative positioning:

2.8.1 Point Positioning

The coordinates of a single point are determined by using a single receiver with code pseudo-ranges .The concept of point positioning is trilatiration in space .For point positioning GPS provides two levels of services:

(1) Standard Positioning Service (SPS) which the C/A code is available and real time accuracies with SA turned off are about 10 m at 95% probability level.

(2) Precise Positioning Service (PPS) has access to both codes and accuracies down to the meter level which the PPS are available only to the

U.S.military .Instead of "single point positioning" term we can use "absolute point positioning" term. [Hoffmann-Wellenhof (2000)]

To determine the receiver point position at any time, a minimum of four ranges to four satellites are required. The receiver gets the satellite coordinates through the navigation message, while the ranges are obtained from either the C/A-code or the P(Y)-code, depending on the receiver type (civilian or military). The measured pseudo ranges are contaminated by both the satellite and receiver clock synchronization errors. Correcting the satellite clock errors may be done by applying the satellite clock correction in the navigation message; the receiver clock error is treated as an additional unknown parameter in the estimation process. This brings the total number of unknown parameters to four: three for the receiver coordinates and one for the receiver clock error. This is the reason why at least four satellites are needed (see Figure 2-15). [Hoffmann-Wellenhof (2000), El-Rabbany Ahmed (2002)]

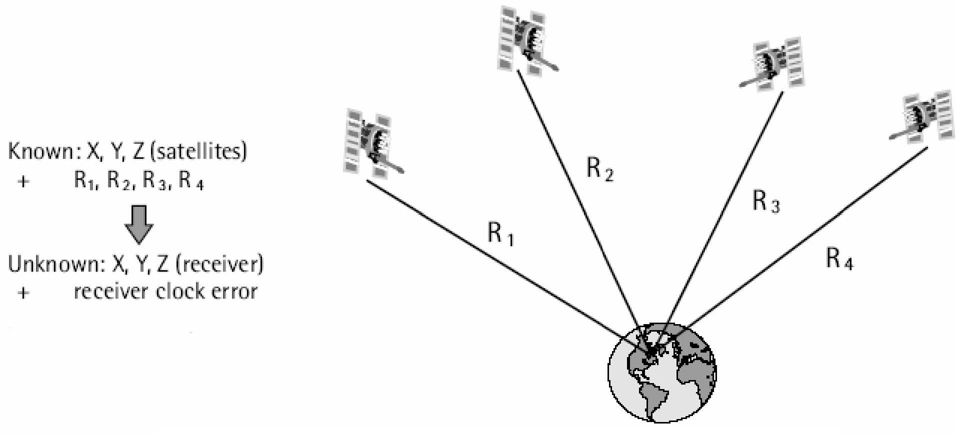


Fig. (2-15) Principal of Point Positioning.

El-Rabbany Ahmed (2002)

2.8.2 Relative Positioning

GPS relative positioning, also called differential positioning (DGPS), employs two GPS receivers simultaneously tracking the same satellites to determine their relative coordinates (see Figure 2-16). The two receivers, one is selected as a reference, or base, which remains stationary at a site with precisely known coordinates. The other receiver, known as the rover or remote receiver, has its coordinates unknown. The rover receiver may or may not be stationary, depending on the type of the GPS operation (static or kinematics). A minimum of four common satellites is required for relative positioning. However, tracking more than four common satellites simultaneously would improve the precision of the GPS position solution. Carrier phase and/or pseudo-range measurements can be used in relative positioning. A variety of positioning techniques are used to provide a post processing (post mission) or real-time solution. GPS relative positioning provides a higher accuracy than that of point positioning.

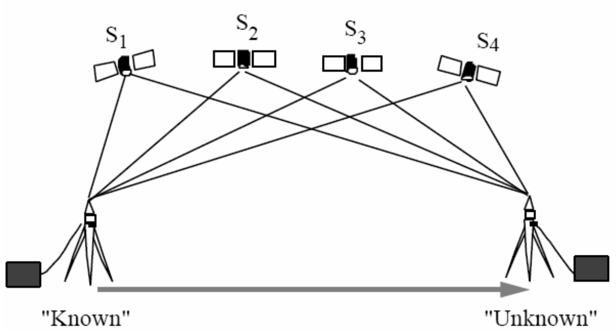


Fig. (2-16) Principal of Relative Positioning.

[Natural Resources Canada (1995)]

The accuracy requirements of GPS users are very different, a very large group of users is interested in a real-time accuracy at a meter level .this accuracy cannot be obtained by point positioning with SPS but can be achieved by DGPS .Using C/A- code range , accuracies of 1-5 m can be achieved . A high accuracy level can be achieved by using carrier phases .For ranges up to 20 km; accuracies of a sub-decimeter level can be obtained, to resolve the ambiguities on -the - fly .[Hoffmann-Wellenhof (2000)]

2.8.2.1 Static GPS surveying

Static GPS surveying is a relative positioning technique that depends on the carrier-phase measurements. It employs two (or more) stationary receivers simultaneously tracking the same satellites (see Figure 2-16). One receiver, the base receiver, is set up over a point with precisely known coordinates. The other receiver, the remote receiver, is set up over a point whose coordinates are sought (the unknown point). The base receiver can support any number of remote receivers, as long as a minimum of four common satellites is visible at both the base and the remote sites. [El-Rabbany Ahmed (2002)]

The static surveying method is the most commonly used, this method requires observation periods depending on the baseline length, the number of visible satellites, the geometric configuration, and the method used. Static relative and also rapid static technique based on fast ambiguity resolution techniques on base lines up to 20 km with various sessions; there sessions depend on satellite visibility, good geometry, and normal atmospheric conditions (see Table 2-8).

Table (2-8) Session lengths for Static Surveys.

[Hoffmann-Wellenhof (2000)]

Receiver	Conventional Static	Rapid Static
L1	30 minutes+ 3minutes/km	20 minutes+ 2minutes/km
L1+L2	20 minutes+ 2minutes/km	10 minutes+ 1minutes/km

2.8.2.2 Kinematic GPS surveying

Kinematic surveying is the most productive to have the greatest number of points in least time .The drawback of this technique is the continuous lock of at least four satellites. To solve the resolution of the phase ambiguities, survey must start with initialization; this initialization can be performed by static or kinematic techniques. The coordinates of the surveying points obtained after finish the surveying in the field as a post-processing.

- Stop-and-Go GPS surveying (Semi-kinematic)

Stop and Go is characterized by stopping and moving one receiver to determine the positions of fixed points along the trajectory. In this technique the relative positional accuracies at centimeter level for baselines up to 10 km.

- kinematic GPS surveying (Rover car)

This technique is best for wide open areas (cleared from obstruction, trees and construction sites). [Hoffmann-Wellenhof (2000)]

- RTK GPS

RTK surveying is a carrier phase-based relative positioning technique that, like the previous methods, employs two (or more) receivers simultaneously tracking the same satellites (see Figure 2-17). This method is suitable when: (1) the survey involves a large number of unknown points located in the vicinity (base length up to about 10-15 km) of a known point; (2) the coordinates of the unknown points are required in real time; and (3) the line of sight, the propagation path, is relatively unobstructed. [Langley, R. B. (1993)b]

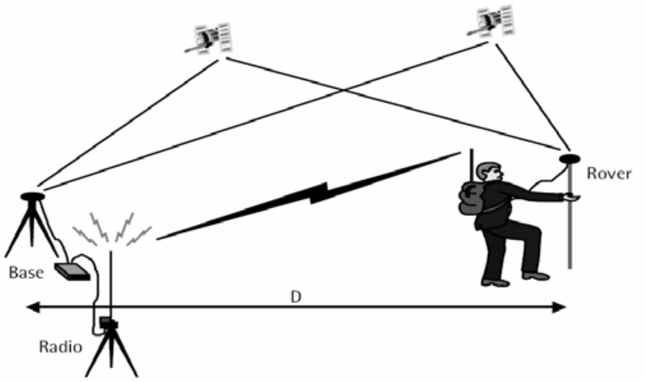


Fig. (2-17) Principal of RTK.

[El-Rabbany Ahmed (2002)]

In this method, the base receiver remains stationary over the known point and is attached to a radio transmitter (see Figure 2-17). The rover receiver is normally carried in a backpack and is attached to a radio receiver. The base receiver measurements and coordinates are transmitted to the rover receiver through the communication (radio) link. The initial ambiguity parameters are determined almost instantaneously using a technique called on-the-fly (OTF) ambiguity resolution. Once the ambiguity parameters are fixed to integer values, the receiver will display the rover coordinates right in the field. That is, no post-processing is required. [Hoffmann-Wellenhof (2000)]

2.9 Glonass

The Global Navigation Satellite System (GLONASS) is based on a constellation of active satellites which continuously transmit coded signals in two frequency bands, which can be received by users anywhere on the Earth's surface to identify their position and velocity in real time based on ranging

measurements. GLONASS and GPS are the same principles in the data transmission and positioning methods. GLONASS is managed by the Russian Federation Government by the Russian Space Forces and the system is operated by the Coordination Scientific Information Center (KNITs) of the Ministry of Defense of the Russian Federation. [Spaceandtech (2009)]
The purpose of the Global Navigation Satellite System GLONASS is to provide unlimited number of air, marine, and any other type of users with all-weather three-dimensional positioning, velocity measuring and timing anywhere in the world or near-earth space.

GLONASS designed constellation is composed of 24 satellites in three orbital planes whose ascending nodes are 120° apart. 8 satellites are equally spaced in each plane with argument of latitude displacement 45°. The orbital planes have 15°-argument of latitude displacement relative to each other. The satellites operate in circular 19100-km orbits at an inclination 64.8°, and each satellite completes the orbit in approximately 11 hours 15 minutes. The spacing of the satellites allows providing continuous and global coverage of the terrestrial surface and the near-earth space.
GLONASS satellites provide two types of navigation signals in the L1 and L2 sub-bands: standard accuracy signal and high accuracy signal.

- The standard accuracy signal with clock rate 0.511 MHz is designed for using by civil users worldwide.
- The high accuracy code with clock 5.11 MHz is modulated by special code and its unauthorized use (without permission of Ministry of Defense) is not recommended. [GLONASS ICD (2001)]

The ground control segment of GLONASS is entirely located within former Soviet Union territory. The Ground Control Center and Time Standards is located in Moscow and the telemetry and tracking stations are in St. Petersburg, Ternopol, Eniseisk, Komsomolsk-na-Amure. [Spaceandtech (2009)]. The current Glonass constellation (14/4/2009) contains 20 satellites (18 operational and 2 in maintenance) [Glonass, (2009)].

2.10 Galileo

Galileo is the future European global navigation satellite system, providing a highly accurate, guaranteed global positioning service under civilian control. It is inter-operable with GPS and GLONASS, the two other global satellite navigation systems. [GALILIO OS SIS ICD (2006)]
GALILEO is the first satellite positioning and navigation system designed for civilian purposes, will be more advanced, more efficient and more reliable than GPS which is controlled by military administration and currently has a monopoly. GALILEO will also help to meet the radio navigation needs in future years to come which it is anticipated can not be satisfied with a single system, where GPS and GLONASS were designed for military purpose. [GALILEO (2008)]

The Galileo constellation consists of 30 satellites (27 operational + 3 spares), positioned in three circular Medium Earth Orbit (MEO) planes at a nominal average orbit semi-major axis of 29601.297 Km, and at an inclination of the orbital planes of 56 degrees with reference to the equatorial plane. Once this is achieved, the Galileo navigation signals provide a good coverage even at latitudes up to 75 degrees north and 75 degrees south.

The Galileo Navigation Signals are transmitted in the four frequency bands, these four frequency bands are: the E5a band, the E5b band, the E6 band and the L1 band. They provide a wide bandwidth for the transmission of the Galileo Signals. [GALILIO OS SIS ICD (2006)]

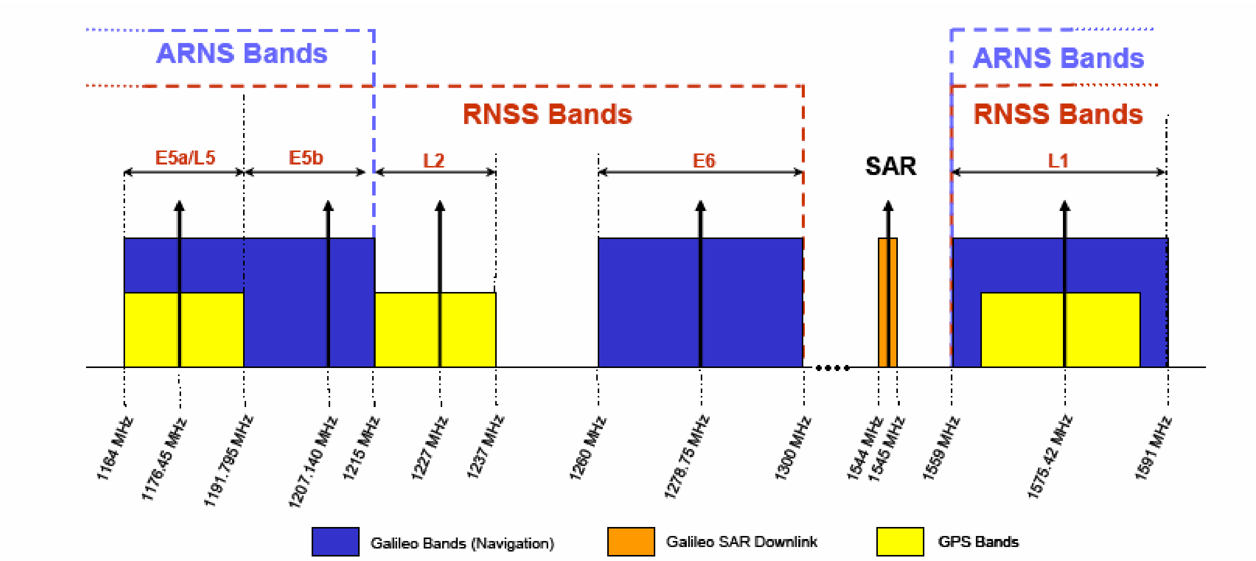


Fig. (2-18) Galileo Frequency Plan.

[GALILIO OS SIS ICD (2006)]

A ground segment is managing the constellation of navigation satellites, controlling the functions of satellite orbit determination and clock synchronization and determining and disseminating the integrity information. It will also provide interfaces with service centers providing value-added commercial services and with the COSPAS- SARSAT ground segment for the provision of search and rescue services.

The first launch of Galileo experimental satellite GIOVE-A was by the end of 2005. GIOVE-B the second Galileo experimental satellite was launched on 27 April, 2008. Both satellites are aimed to test and validate the reception and performance of novel code modulations designed for Galileo including new signals based on the use of the BOC (Binary Offset Carrier) technique, in particular the high-performance E5AltBOC signal. Galileo is aimed to be fully functional by 2012.

2.11 Datum and Map Projection of GPS Surveying

2.11.1 Geodetic Datum

To determine the precise location of a user anywhere, under any weather conditions, to attract millions of users worldwide from various fields and backgrounds; so there must be a datum for GPS surveying to tie between all users [El-Rabbany Ahmed (2002)]. Datums can be defined as surfaces or sets of quantities upon which measurements are based. Two different types of datums must be considered: vertical datums and horizontal datums . In fact, the topographic surface of the Earth is highly irregular makes it difficult for the geodetic calculations, to overcome this problem, geodesists adopted a smooth mathematical surface, called the reference surface, to approximate the irregular shape of the earth. However, the best mathematical surface to approximate the Earth and at the same time keep the calculations as simple as possible was found to be the biaxial ellipsoid (see Figure 2.19). The biaxial reference ellipsoid or simply the reference ellipsoid is obtained by rotating an ellipse around its minor axis, b. Similar to the ellipse, the biaxial reference ellipsoid can be defined by the semi-minor and semi-major axis (a, b) and the flattening (f), where f = 1 – (b / a). [El-Rabbany Ahmed (2002)]

The geodetic datum is geocentric datum with its origin coinciding with the center of the Earth, It is clear that there are an infinite number of geocentric geodetic datums with different orientations. The common global reference system called WGS-84(World Geodetic System). [Leick (1995)]

WGS-84 is a terrestrial reference frame which realized by the coordinates of about 1500 terrestrial sites which derived from TRANSIT observations ,WGS-84 has the following parameters that show in Table (2-9).

Table (2-9): Parameters of WGS-84.

[Hoffmann-Wellenhof (2000)]

Parameters and Value	Description
a= 6378137.0 m	Semi-major axis of the ellipsoid
1/f = 298.257223563	Flattening of the ellipsoid
$\omega E = 7292115*10^{-11}$ rad s^{-1}	Angular velocity of the earth
$\mu = 3986004.418*10^{8}$ m^{3} s^{-2}	Earth's gravitational constant

The coordinates of any point in WGS-84 represents in ellipsoidal coordinates (φ, λ, *h*) but can be transformed to Cartesian coordinates (X, Y, Z) (see Figure 2.20) .[Hoffmann-Wellenhof (2000)]

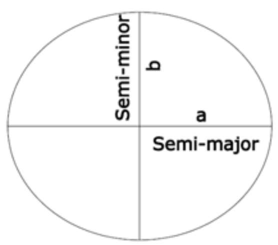


Fig. (2-19) Ellipse Components

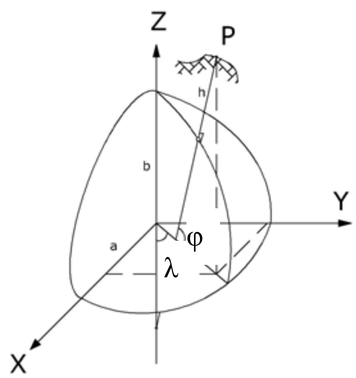


Fig. (2-20) Cartesian and Ellipsoidal Coordinates.

2.11.2 Height Reference Datum

The vertical datum of the geodetic datum is a reference to compute the elevation (height) of earth point, where the vertical reference datum has a zero height so any point height measured from datum called ellipsoidal height but the Orthometric height measured from the geoid [Leick (1995)] (show Figure 2-21).

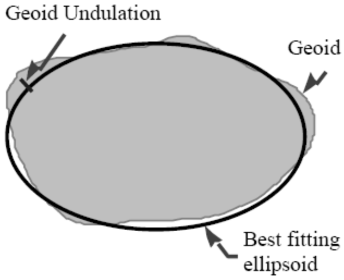


Fig. (2-21) Geoid and Ellipsoid.

[Natural Resources Canada (1995)]

The geoid is the mean sea level around the earth; the geoidal surface (geoid model) is very irregular and virtually impossible to realize an exact mathematical model for the geoid [Hoffmann-Wellenhof (2000)]. The distance separating the geoid and the ellipsoid is the *geoid undulation N* (also called geoid height). The geoid undulation may be positive or negative depending on whether the geoid is above or below the ellipsoid at a given point (see Figure 2-21). The Orthometric height may be determined using the relationship as in equation (2.9). [Hoffmann-Wellenhof (2000), Natural Resources Canada (1995)]

$$H = h - N \qquad (2.9)$$

Where:

H: The Orthometric height.

h: The ellipsoidal height.

N: Geoid undulation.

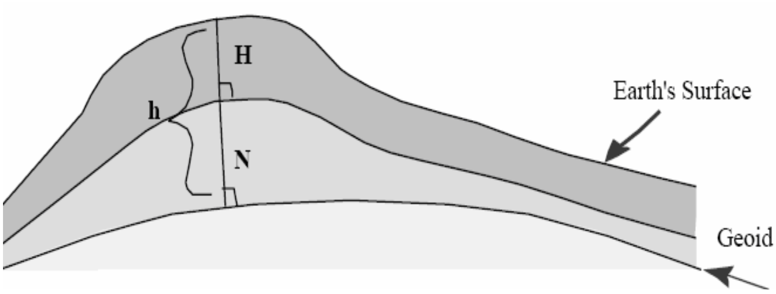


Fig. (2-22) Relationship between Orthometric and Ellipsoidal Heights.

[Natural Resources Canada (1995)]

2.11.3 EGM-96 Geoid Model

The NASA Goddard Space Flight Center (GSFC), the National Imagery and Mapping Agency (NIMA), and the Ohio State University (OSU) have collaborated to develop an improved spherical harmonic model of the Earth's gravitational potential to degree 360.

Earth Gravitational Model 1996 (EGM96) incorporates improved surface gravity data, The final solution blends a low degree combination model to degree 70, a block-diagonal solution from degree 71 to 359, and a quadrate solution at degree 360.

This model was used to compute geoid undulations accurate to better than one meter (with the exception of areas void of dense and accurate surface gravity data) and realize WGS84 as a true three-dimensional reference system. [Lemoine et al. (1998), Uliana Danila (2006)]

2.11.4 Local Coordinate System

The official reference system of GPS is WGS-84 but in the terrestrial sites; the surveyors are not interested in global coordinates, so the results are preferred in local coordinates. This local system is transformed from the global system with transformation parameters (see Figure 2-23). [Hoffmann-Wellenhof (2000)]

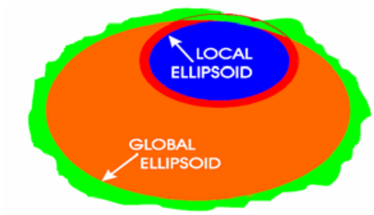


Fig. (2-23) Global Ellipsoid and Local Ellipsoid
[EUROCONTROL (1998)]

The transformation parameters between the two geodetic systems (global and local) are defined by 9 parameters as follow:

- Translation of the origin ΔX, ΔY, ΔZ
- Rotation angles εx, εy, εz
- Scale factor μ
- change in ellipsoidal semi-major axis Δa and flattening Δf

The 9 parameters define the location and orientation of a (local) coordinate system with respect to the global reference frame. [EUROCONTROL (1998)]

2.11.5 Map Projections

Map projection is defined from the geometrical view; as the transformation of the physical features on the curved Earth's surface into a flat surface called a map (see Figure 2-24). However, it is defined; from the mathematical view, as the transformation of geodetic coordinates (φ, λ) obtained from, for example, GPS, into rectangular grid coordinates often called easting and northing. [El-Rabbany Ahmed (2002)]

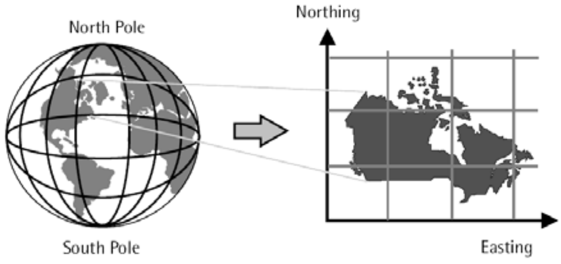


Fig. (2-24) Concept of map projection.

[El-Rabbany Ahmed (2002)]

2.11.5.1 Conical Projection

This projection is as a cone which a tangent of the ellipsoid at a selected standard parallel, the meridian are straights lines converging at a point called the apex. An example of this projection is Lambert Projection (see Figure 2-25).

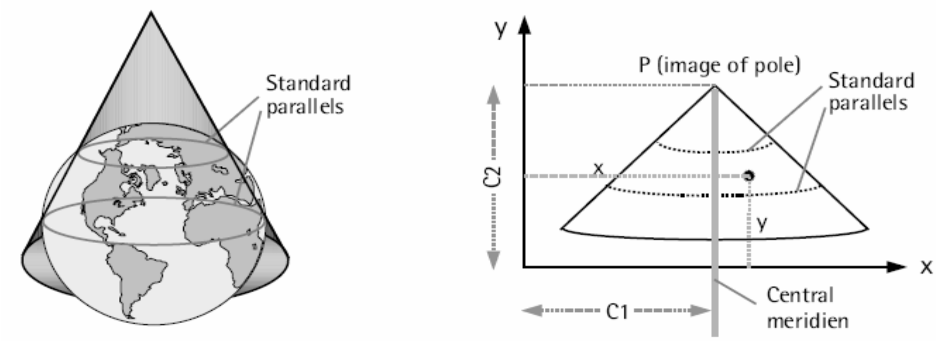


Fig. (2-25) Lambert Conical Projection.

[El-Rabbany Ahmed (2002)]

2.11.5.2 Cylindrical Projection

This projection is a special case of conical projection if the apex point is moved to infinity so the cone becomes a cylinder which is a tangent to the equator in the transverse position the cylinder is a tangent to the standard meridian, after this cylindrical surface the standard meridian mapped without distortion .two examples of cylindrical projection are the Transverse Mercator (TM) Projection and Universal Transverse Mercator (UTM) Projection.

- Transverse Mercator Projection (TM)

This projection called also Gauss- Krűger projection .the ellipsoid is partitioned into 120 zones of 3 degrees longitude each where the central meridian (CM) is in the center of each zone .the central meridian of zones is mapped onto the plan without any distortion parallels to the north direction (Y axis) and the x- axis is the equator (see Figure 2-26). [Hoffmann-Wellenhof (2000)]

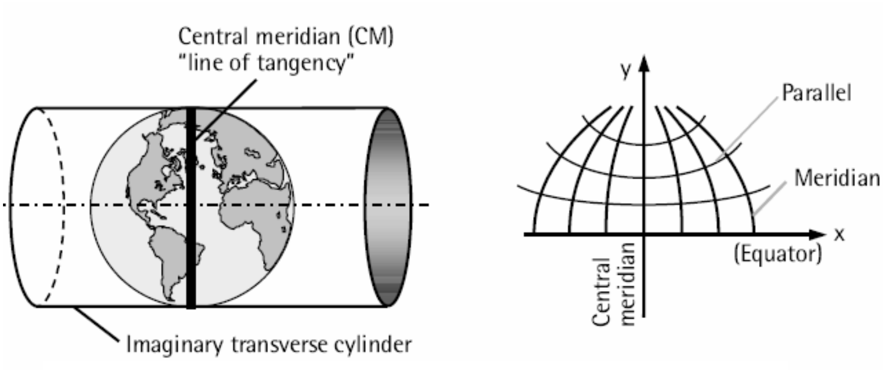


Fig. (2-26) Transverse Mercator projection.
[El-Rabbany Ahmed (2002)]

- Universal Transverse Mercator Projection (UTM)

UTM is the modification of TM projection; the ellipsoid is partitioned into 60 zones with a width of 6 degrees longitude and with a scale factor of 0.9996 this scale factor is applied to the conformal coordinates in the plane, the reason of this scale factor is to avoid fairly large distortions in the outer areas of a zone. The zone numbering starts with M1 for CM λ_0=177° W and M2 = λ_0=171°, the zone number can be calculated from the next formula

$$\mathrm{INT}\left(\frac{180 \pm \lambda}{6}\right) + 1 \qquad (2\text{-}10)$$

Where (+) sign for eastern longitude and (-) sign for western longitude. [Hoffmann-Wellenhof (2000)]

To avoid negative coordinates, the true origin of the grid coordinates (where the equator meets the central meridian of the zone) is shifted by introducing the so-called false northing and false easting. The false northing and false easting take different values, values, depending on whether we are in the northern or the southern hemisphere. [El-Rabbany Ahmed (2002)]

As an example, for Egypt; the UTM zone is 36N that have the following parameters:

- Datum: WGS-84.
- Projection: Transverse Mercator.
  - Latitude of Origin= 0 ° N
  - Central Meridian= 33 ° E
  - Scale factor= 0.9996
  - False Easting= 500000.00 m
  - False Northing= 0.00 m [GNSS Solution (2007)]

2.11.5.3 Stereographic Projection

This projection is a special case from the conical projection if the apex point moved to the pole so the cone becomes a plane which is a tangent at the pole; the plane projection can be defined as a plane tangent at any point on the ellipsoid .an example of this projection the stereographic projection (see Figure 2-27). [Hoffmann-Wellenhof (2000)]

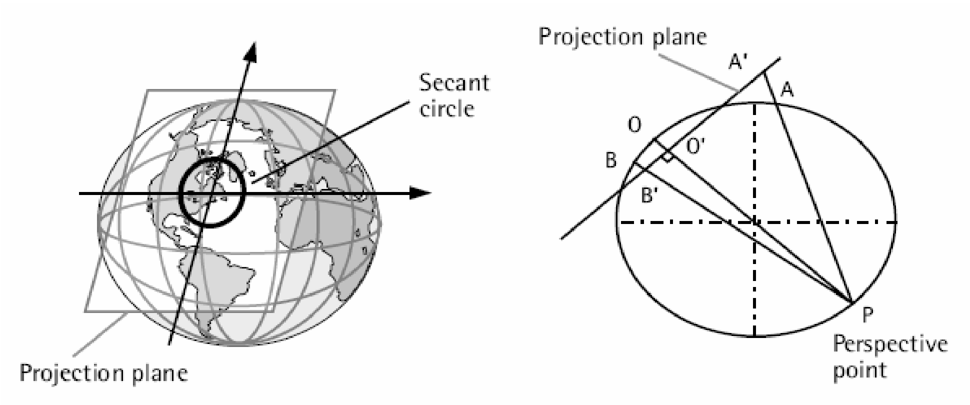


Fig. (2-27) Stereographic Projection.

[El-Rabbany Ahmed (2002)]

2.12 <u>Digital Elevation Model (DEM)</u>

DEM defines as the solution model in showing of land characters and appearances, DEM aims of evaluating data collected from land surface in computer. DEM first appeared in 1950 to use for using calculation of width and length appearance, volume in road project and applied for models including latitude data. Later in addition to latitude data, it was understood that it is necessary to evaluate natural and artificial details of land in DEM. [Yılmaz Ibrahim, et al(2006)]

DEM have also been use in many areas such as industry, medicine, architecture, mining, agriculture, and mapping. There are many other terms of DEM such as the Digital Terrain Model (DTM), Digital Height Model

(DHM), Digital Ground Model (DGM), and Digital Terrain Elevation Data (DTED). [Othman M. Awad (2005)]

There are many applications of creating DEM,

- Civil Engineering: civil engineers are mainly interested in using DEMs to compute cut-and-fill involved with road design, in site planning and volumetric calculation in building, dams and reservoirs. Earth Sciences: the earth or geo-scientific applications mainly center on specific functions modeling, analysis and interpretation of the unique terrain morphology. These may include drainage basin network development and delineation, hydrological run-off modeling, geomorphologic simulation and classification, and geological mapping. Generating slope and aspect maps, and slope profiles for creating shaded relief maps are popular tasks performed in the earth sciences that employ DTMs.
- Planning and Resources Management: this is a major grouping of diverse fields including remote sensing, agriculture, soil science, meteorology, climatology, environmental and urban planning, and forestry, whose central focus is the management of natural resources.
- Surveying and Photogrammetry: one of the main objectives of employing surveying and photogrammetry is in building reliable DTMs, evaluating their accuracy towards finally producing high quality contours. This may be done subsequent editing, ortho-photos production, data quality assessment, and topographic mapping.
- Military Applications: the military domain is not only a leading consumer of DTMs; they also are a significant producer. Almost every

aspect of the military environment depends on reliable and accurate understanding of the terrain, elevation and slope of the land surface. The military usage of DTMs combine facets and methods of all previous application domains, and their and objectives are very specialized and demanding.[J. R. Sulebak, (2000)]

DEM creates by many surveying methods such as

- Graphics Digitizing of topographic printed map.
- Ground Survey methods.
- Photogrammetric and remote Sensing.
- DEM data from airborne profile record (APR).
- LIDAR (Light Detection and Ranging).
- GPS Techniques.

Chapter 3

GPS Accuracy Assessment

GPS Accuracy Assessment

3.1 Introduction

It is important to be aware of the various terms used to quantify measurement accuracies and the relationship between them so that GPS accuracy claims may be compared, to have the goodness of the point position and its coordinates.

This chapter aims to show the accuracy assessment of GPS with types of errors, GPS and Digital Elevation Model accuracies and show the previous work in accuracy assessment.
This chapter contains of the following categories

- Types of Errors.
- Precision and Accuracy.
- Probability Models
- Factors affecting on Accuracy of GPS.
- Previous GPS Accuracy Assessment Studies for Static & Kinematic Techniques.
- Previous GPS Accuracy Assessment Studies For DEM

3.2. Types of Errors

Errors can be classified to three types as shown

3.2.1. Gross Errors

Gross errors are the results of blunders or mistakes that are due to carelessness of the observer. For example, reading the wrong scale, make point on wrong survey target, reading the recording number in wrong way (read 41.56 instead of 41.65) and insert the GPS antenna height above the point 0.5 m instead of 1.50 m. These errors can't be tolerated so they must be detected and corrected.

To avoid these mistakes, good field procedures are designed to assist detecting mistakes. These procedures include:

- Careful checking of all points on survey targets.
- Tacking multiple reading on scales and checking for reasonable consistency.
- Verifying inserted data to the GPS receiver.

3.2.2. Systematic Errors

Systematic Errors are those which have some known pattern or behavior which biases the observations. Systematic errors are removed from observations by modeling. For example, much of the error due to the tropospheric delay, it may be removed by applying a mathematical model which represents tropospheric behavior. [Mikhail, E (1976)]

3.2.3. Random Errors

After all blunders are detected and removed, and the measurements are corrected from all known systematic errors, there will still remain some variation in the measurements. This variation results from observational errors which have no known functional relationship based upon a deterministic system. These errors have random behavior and themselves are random variables. Random errors are dealt with mathematically using functional relationship or models called *probability models*.

3.3. Precision and Accuracy

3.3.1. Precision

Precision is the degree of closeness or conformity of repeated measurements of the same quantity of each other. If the repeated measurements are closely clustered together, they are said to have high precision; if they are widely spread apart, they have low precision. High precision generally reflects the high degree of care and refinement in the instrumentation and procedure used in making the measurements. Precision is dispersion or spread of the indicated by the probability distribution. The narrower distribution, the higher precision, and vise versa. A common measure of precision is the standard deviation σ. The higher precision, the lower is the value of σ, and vise versa.

3.3.2. Accuracy

Accuracy is the degree of conformity or closeness of a measurement to the true value. Accuracy includes not only the effects of random errors but also any bias due to uncorrected systematic errors. If there is no bias, the standard deviation can also be used as a measure of accuracy. [Mikhail, E.M, Gordon Gracie. (1981)]

Precision includes only random effects, while accuracy includes both random and systematic effects, see Figure (3-1). [Mikhail, E.M. (1976)]

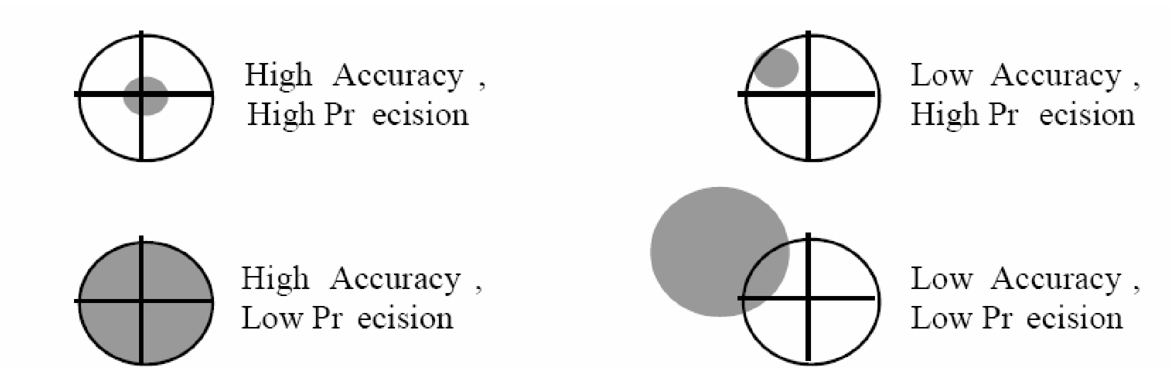


Fig. (3-1)Accuracy and Precision.

[Natural Resources Canada (1995)]

3.3.3. Absolute and Relative Accuracy

3.3.3.1. Absolute Accuracy

Absolute accuracies are estimates of how close a position is to the truth in the earth's reference frame, there are always represented as constant values.

3.3.3.2. Relative Accuracy

Relative accuracies may be represented as constant values, as parts per million (ppm), or both. Parts per million are used to relate error magnitudes with baseline length. For example 1 ppm corresponds to a 1 mm error over 1 km and a 1 cm error over 10 km. The linear relationship between errors and baseline distance for 2, 10 and 20 ppm is shown in Figure (3-2)

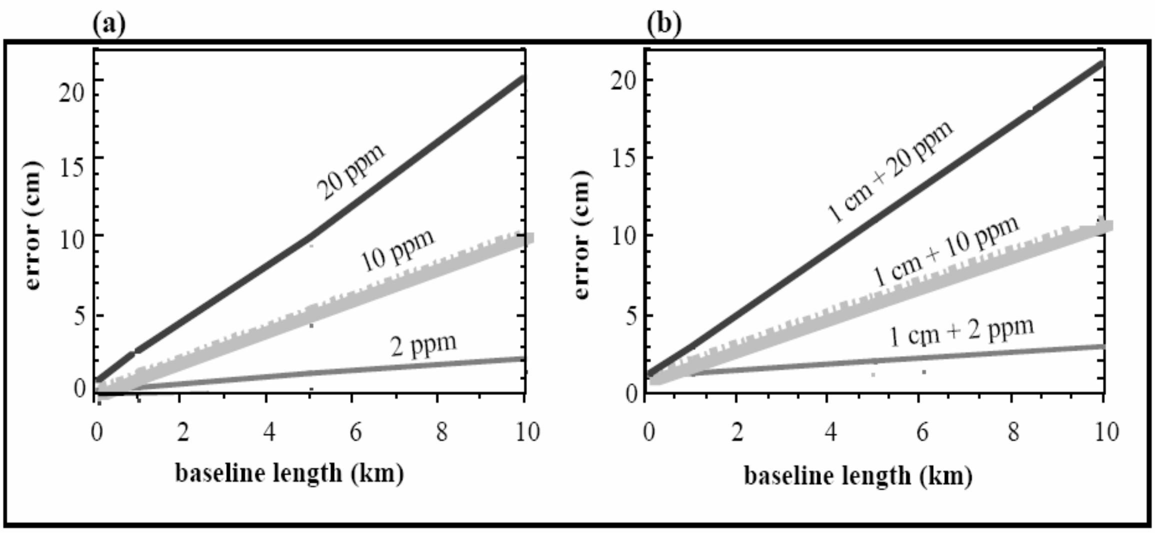


Fig. (3-2) GPS Relative Accuracies - PPM a) ppm b) constant + ppm.

Natural Resources Canada (1995)

Baseline accuracies using GPS are often expressed combining a constant term (e.g. 1cm) with a linear term (e.g. 1 ppm). For example, the accuracy of a precise survey could be specified as:

$$\text{RMS accuracy} = 1\ \text{cm} + 1\ \text{ppm}. \qquad (3\text{-}1)$$

The same relationship shown for ppm in Figure (3.2a) is shown in (3.2b) with the addition of a 1 cm constant. In GPS, the constant term accounts for errors which are independent of baseline length such as antenna set-up and multipath errors, while the linear term accounts for length dependent errors such as residual orbital, tropospheric and ionospheric errors. [Natural Resources Canada (1995)]

The relative accuracy of GPS surveys depends mainly on baseline length, session duration, methodology and the software used for processing GPS data [Eckl et al, 2001]

3.4. Probability Models

Random errors have the property that if enough observations are made there will be equal probability of negative and positive errors, yielding a mean value of zero. Random errors, according to statistical theory, tend to be distributed about the mean following *the normal probability distribution function* (Figure 3.3)The area under the curve represents all potential random error outcomes according to the theory of *normal distribution*.

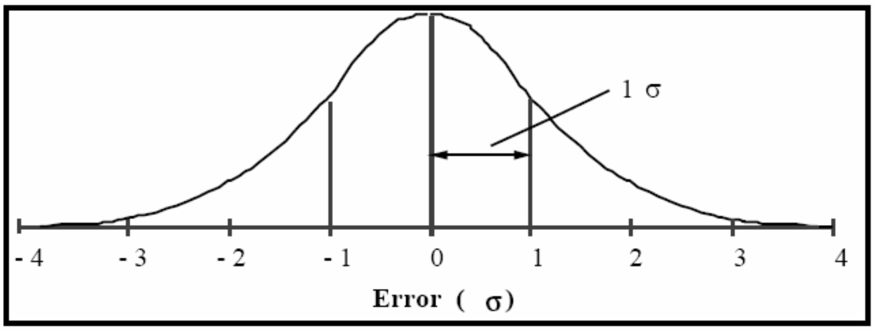


Fig. (3-3)Normal Probability Distribution Function.

Natural Resources Canada (1995)

The *standard deviation*, represented by the symbol σ, is used to quantify dispersion about the mean and is shown on the normal probability distribution function (Figure 3-3) the certainty of a solution may be quantified by multiples of the standard deviation or by probability. The normal probability distribution function gives the relationship between the two. For example, a standard deviation of 1σ is associated with a probability of 68.3% (the percent of the area under the curve in Figure (3-3) bounded by ±1) and a 95% probability is associated with 1.96σ [Mikhail, E.M. (1976)].

Table. (3-1) Relationship between Standard Deviation and Probability - 1D Case.

[Mikhail, E.M. (1976)]

Multiples of σ	Probability	Probability	Multiples of σ
1 σ	68.27%	90%	1.645 σ
2 σ	95.45%	95%	1.960 σ
3 σ	99.73%	99%	2.576 σ

The Standard Deviation (σ) indicates the accuracy of a set of measurements relative to the mean value (the average). Mathematically expressed as:

$$\sigma = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(v)^2} \qquad (3\text{-}2)$$

Where:

σ is the standard deviation (standard error) .

v= (M-M') the residual of any measurement

M is any measurement.

M' is the most probable value (mean).

n is numbers of measurements. [Russell C. Brinker, etc (1994)]

Note that the 50% error is called *probable error*.

The another term to describe the errors measurements is RMSE

$$\text{RMSE} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(v')^2} \qquad (3.3)$$

Where:

RMSE is the Root Mean Square Error.

$v' = (M - M_o)$ the residual of any measurement

M is any measurement.

M_o is the original value of measurement.

n is numbers of measurements.

3.5. Factors affecting Accuracy of GPS

The accuracy of the GPS positioning depends on two main factors:

3.5.1. Measurement accuracy

Measurement accuracy expressed by the User Equivalent Range Error (U.E.R.E), that depending on

- Effect of ephemeris uncertainties.
- Propagation of errors.
- Clock and timing error.
- Receiver noise.(see (2.7.2))

3.5.2. Satellite Configuration geometry

Satellite Configuration geometry expressed by Dilution Of Precision (D.O.P), where D.O.P is the ratio of the positioning accuracy to the measurement accuracy.

$$\sigma 1 = \text{D.O.P } \sigma o \qquad (3.4)$$

$\sigma 1$ the position accuracy.

σo the measurement accuracy.

D.O.P = σ/ σo

For minimum D.O.P., the best configuration as follow: see figure (3-4)

- One satellite over head, 3 satellites on horizon, separated by 120º in azimuth, with elevation angle of 20º.
- The four satellites are uniformly distributed on the horizon.[Natural Resources Canada (1995), Antoun Nagi F. (1998)]

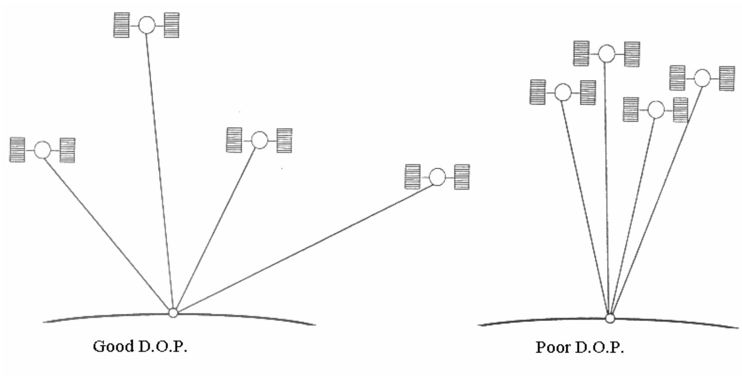


Fig. (3-4) Good and Poor D.O.P.

There are types of D.O.P., see table (3-2)

Table. (3-2)D.O.P. Types.

[Natural Resources Canada (1995)]

ID	Type	Position Components
G.D.O.P.	Geometrical	3D position and time
P.D.O.P.	Position	3D position
H.D.O.P.	Horizontal	2D horizontal position
V.D.O.P.	Vertical	1D height

For GPS positioning, the lower the PDOP is the better. A PDOP below 5 or 6 is generally the recommended upper limit for positioning, particularly for short occupation times (e.g. a few minutes). For static relative positioning over long time periods, (e.g. more than one hour) the PDOP is not quite as critical since one benefits not only from the geometry of the satellite configuration, but also from the geometry of the path the satellites trace in the sky over time. [Natural Resources Canada (1995)]

3.6. Previous GPS Accuracy Assessment Studies For Static & Kinematic Techniques

Many studies were published for GPS Accuracy Assessment and related factors affecting on accuracy for static and kinematic techniques as follow:

- Dalal S. Alnaggar, Gomaa M. Dawod (1999)

This research show producing a topographic map for area equal to 75000 acres in Toshka region in south valley development project. This work consists of three methods of observations, static GPS, OTF relative kinematic GPS, leveling. This GPS technique reduces the needed time with 75% and 50% of costs of applying the traditional surveying methods.

- Shaheen Bahaa . (2001)

This study aims to create the Greater Cairo Geodetic Network (GCGN) by using GPS Static Technique. The results of this study as follow

  - The standard deviation values of X direction is greater than the other Cartesian components (Y and Z).
  - Using the almanac data file is important to give the suitable time of observation with good satellite elevation and good PDOP values.

- Increasing the period of time of observation about four times increases the accuracy by about 50%.
- Precise ephemeris data gives higher accuracy than Broadcast ephemeris data. For short baselines the difference between precise and Broadcast ephemeris data is about 5 to 10 %, but for long baselines the different between them is very clear in about 50 to 75%.
- Lowering the elevation angle improves the precision and decreases the standard deviation .The best elevation angle ranges from 15 to 25 degrees.
- Changing the interval time between epochs has no effect in standard deviation values for interval time 15 and 30 seconds.
- For baselines rounded to 100 kms use only 3 hours sessions to save time, because of increasing time session more than 3 hours doesn't affecting the accuracy and the coordinates of stations will change about only few millimeters.

- Waypoint Consulting, 2001

It is proposed that 60 minutes of observation using L1 carrier phase measurements with fixed solution is required to obtain 0.50-2 cm accuracy when observing a baseline of lengths up to 5.0 km, see Table (3-3)

Table (3-3): L1 Carrier Phase Fixed Solution Typical accuracies and required Observation session, [after Waypoint, (2001)]

L1 Carrier Phase Fixed Solution Typical Accuracies with 5 Satellites		
Baseline Length	Observation Time at Remote Station	Expected Coordinate Error
0-5 km	5 minutes	N/A
0-5 km	10 minutes	N/A
0-5 km	20 minutes	0.5-2 cm
0-5 km	40 minutes	0.5-2 cm
0-5 km	60 minutes	0.5-2cm

- Lee, J. and Jang, H., (2001)

This study aimed to compare between the Static and Rabid Static techniques by observing baselines of a distance equals to 1km, 2.5km, 5km, 7.5km, 10km, 15km, 20km for Static Technique, The times of survey were 30, 60, 90, and 120 minutes and acquisition space of the data was 30 sec. The distances of Rapid Static Technique were selected 1km, 2.5km, 5km, 7.5km, 10km, 15km as Method of static positioning, and the time of survey were 5, 10, 15, and 20 minutes and acquisition space of the data was 5 sec. the results of this study as follow:

- The proposed time can be reduced with increasing the number of tracked satellites using differential static technique.
- Control point relative observation allowable error as shown in Table (3-4)

Table (3-4): Control Point Relative Observation Allowable Error.

Division	Control Point			
	First Grade	Second Grade	Third Grade	Fourth Grade
Baseline	30~37 (km)	10 (km)	5 (km)	2.5 (km)
Accuracy	1/1,000,000	1/500,000	1/200,000	1/100,000

- El-Shazly A., Abdel-Maguid R. (2004)

This study shows the relative accuracy of observation a baseline equal 27 km, with various sessions (10, 20, 30, 40, 50, 60, 70, 80, 100, 120 min., 3hours), 15 seconds sampling rate and cut off angle 15 degree for satellite elevation angle, The relative accuracy of the point position is calculated depending on assuming that the 6-hr observation is the accurate or reference data, the results of this study as follow:

- Long period of observation may not insure give higher level of accuracy.
- There are great relative variations of rover positional accuracy when changing observation time window within the allowable PDOP values, but increase the session duration, may not raise the accuracy level.
- Increasing of satellite numbers and decreasing PDOP values during the observation, increase the accuracy level of point position.
- The session duration, as a guide rule, should be about 10 minutes + 1 min/km for single frequency receivers.

- Zhou, G., (2004)

He shows in his lecture notes the required relative accuracies for GPS surveys, see Table (3-5).

Table. (3-5) Required relative accuracies for various GPS surveys.

Order	Distance Accuracy	Relative Accuracy	Purpose
A	0.5 cm	0.1ppm	US Geodetic Reference Network, Earth Surface Deformation
B	0.8 cm	1ppm	Local Earth Surface Deformation, high-Accuracy Engineering Surveying
C	1.0 cm	10ppm	Engineering Surveying, Urban Control Surveying

- Ordnance Survey (2004)

It shows that the required observation time may be increased if greater levels of accuracies are required as presented in Figure (3-5)

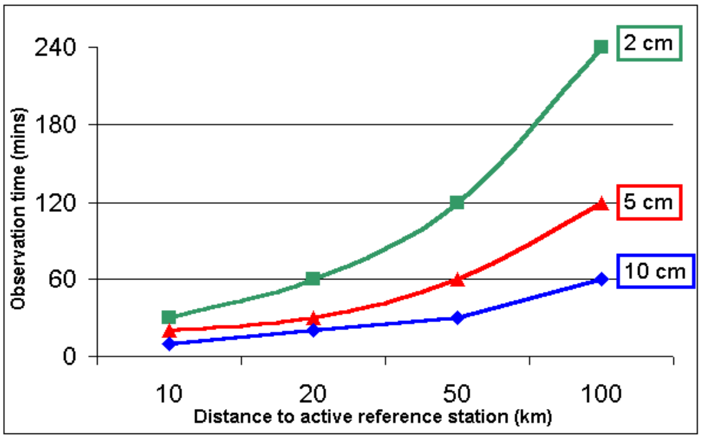


Fig. (3-5) The Required observation time for different level of accuracies for various baseline lengths.

- ABDEL-MAGUID Ramadan H., et al (2005)

This study summarizes various methods used for mapping of small villages in Egypt to save time, money and effort to survey an area of 100 acres. The methodology of this study depends on

- Using GPS Static Technique to do two or more control stations in the whole area of the village to tie the area to the national mapping system, the accuracy of the final control points is less than 1 cm.
- Using the kinematic GPS measurements (OTF) to survey the centerlines of the streets using moving van with receiver antenna of 3 m to reduce satellite loss of lock, the fixed solution was obtained to give the centimeter- level.
- Terrestrial surveys Using total station in the main roads of the area.

- The differences between the two surveying systems (kinematic GPS and total Station) are 0.0 to 0.41 m with a mean of about ±16 cm in Planimetric.

3.7. Previous GPS Accuracy Assessment Studies for DEM

There are many studies about creating Digital Elevation Model (DEM) as follow:

- Khalil, R. (1993):

This research show the factors affecting the accuracy of DEM as follow:

- Specified accuracy requirement.
- Source of data, which includes:

- Equipment and operator precision
- Measuring accuracy
- Sampling density

- Terrain type.
- Sampling pattern.
- Interpolation method.
- Data storage method.

If the DEM was generated using aerial photographs or satellite image addition parameters may be influence the DEM accuracy such as:

- Quality of the photographs or images.
- Scale of the photo (focal length / flying height) or the resolution of the satellite sensor.
- Number, location and accuracy of control points.

- El Wakil Amr M. Mosa (2000)

This thesis is an experimental study to draw a topographic map of an area of 200*200 m by using Total Station and by GPS RTK instrument (Rover and Stop and Go). The Total Station was the reference data for comparison. Total Station measurements were taken randomly on the whole area, The Total Station results not completely free from errors, but there are random errors.

The differences between Total Station and RTK (Stop and Go) range was in between (-9.0 to 7.50 cm) for cross sections and the differences between Total Station and RTK (Kinematic) are in between (-10.4 to 9.80 cm). The research show that performance of GPS was very efficient compared with Total

Station, GPS is more quickly, less efforts, over long period, cheaper, and finally no need to office work, GPS is more economically than Total Station in all faces and RTK GPS save more than 75% of the consumed time to survey the conventional survey.

- Tokmakidis, K., et al (2003), C. Pikridas, et al (2004):

  This study intends to test the accuracy of the reference DTMs produced with classical photogrammetric techniques (produced in the 1970s by the Hellenic Military Geographical Service, with classical photogrammetric techniques) comparing them with DTMs which can derived from other sources, such as ground measurements using GPS and spirit leveling. Elevation data are a very sufficient data source for many engineering applications since they can be used for a long time and represent the landscape of the study area. This comparisons show that the differences between the two techniques of DTM are in from (2 cm) up to 70 cm and the combination of GPS and local geoid is a very efficient method of capturing data with few centimeters accuracy in order to produce a reliable DTM, but it needs a lot of fieldwork for capturing dense elevation data.

- Othman M. Awad (2005):

The objective of this thesis is to study the effect of breakline incorporation on different terrain types using different sampling density and different interpolation methods. In addition, To estimate the economical point spacing for each terrain type suitable for a demand accuracy in case of breaklines incorporation and without incorporation of breaklines. The second objective was to select the best mathematical model for many applications of the DEMs in case of breaklines incorporation and without incorporation of breaklines. This research use Surfer 7.0 package software to draw contour maps and to calculate cut and fill volumes. The research show that

- Te minimum curvature method and kriging method give better results than radial basis function method and inverse distance to a power method for each terrain type. In addition, breaklines incorporation improves DEM accuracy with different rate according to interpolation method used. The effect of breaklines incorporation is biggest on inverse distance to a power interpolation method. and have nearly the same effect in other interpolation methods.

- The interpolation methods that give the best results for the different applications such as generating contour maps, total volume computation, and calculation of the volume of earthwork are the minimum curvature and kriging method with breaklines for different terrain types.
- Uncertainty, instead of error, should be used to describe the quality of a DEM. To analyses the pattern of deviation between two sets of elevation data, conventional ways are to yield statistical expressions of the accuracy, such as the Root Mean Square Error, and Standard Deviation.
- The most widely used measure is the Root Mean Square Error (RMSE). It measures the dispersion of the frequency distribution of deviations between the original elevation data and the DEM data.

- Yılmaz Ibrahim, et al(2006)

In this research, DTM of study area created with Kriging interpolation technique was obtained via classical measurements and GPS (stop&go and kinematic) technique. It shows that DEM elevations using GPS were close to classical measurements. This assign that necessary data to create DTM could be obtained from GPS kiematic technique, the other advantage of GPS is lower number of necessary persons than that of classical methods, this makes big save in time and cost.

EXPERIMENTAL WORK

Introduction

The objective of this chapter is to show the field work of GPS applications on Aswan Governorate; the field work includes two GPS observation types. First type is GPS Static surveying; this surveying type aims to detect the relative positioning accuracy of GPS Static surveying using single frequency receivers. This type is used to create control points in national networks. Second type includes two types of GPS kinematic surveying; the first type is creating a Digital Elevation Model using Stop and Go GPS kinematic technique and compare the results with a Total Station . This type is important to study the possibility of using GPS in creating grids and contour maps for leveling applications for new construction projects and reclamation of desert; where the new direction of Egyptian Government is to increase the populated and green areas and to get out from narrow Nile valley to wide desert areas; especially in Aswan, there are a wide areas a round Lack Nasser behind High Dam. The second type is surveying a road using moving car and compare the results with a total station technique for the same road; this type aims to detect the absolute positioning accuracy against Total Station height; this technique is used in road planning applications.

4.1 Field Planning

Planning is important step on surveying any field work. Field planning includes some steps as follow

4.1.1 Pre-survey Planning and Field Reconnaissance

The first step in planning a GPS Survey is to obtain a largest scale map of the location area to detect the desired points and plotted on the Topographic Map to avoid any obstructions on field and to give the possibility to reach those points. So there are four basic considerations in choosing a survey point:

- No obstructions above 15 ^{o} elevation angles to avoid satellite signal blockage and have a good sky visibility.
- No reflecting surfaces to avoid multipath.
- No nearby electrical installations to avoid signal disturbances.
- No transportation beside the receivers to protect points and the surveyors. [Hoffmann-Wellenhof (2000)]

4.1.2 Mission Planning

Mission planning introduces a representation about the satellite coverage to detect the suitable observation time with high satellite numbers and low GDOP (Geometric Dilution of Precision) to get a high accuracy level, mission planning is running by GNSS Solution software by using an almanac data file of the observation day. This software gives many plots such as polar sky, GDOP and Satellite Visibility plots; see figures (4-1), (4-2), (4-3). [GNSS Solution (2007)]

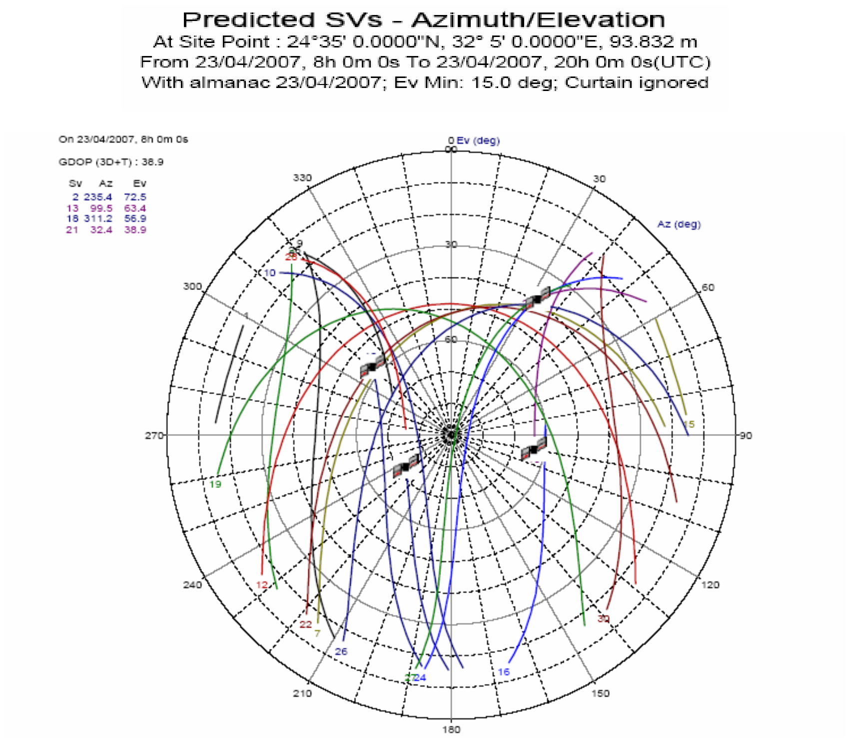


Fig (4-1): Polar sky plot of elevation angle of 15º for 23-4-2007 .

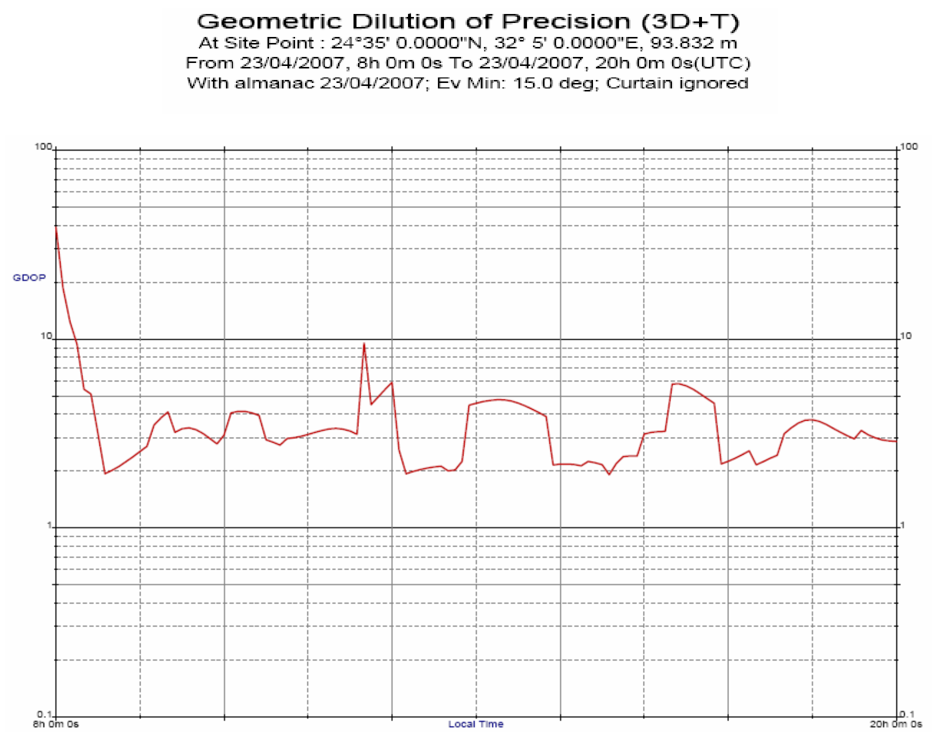


Fig (4-2): GDOP plot of elevation angle of 15º for 23-4-2007 .

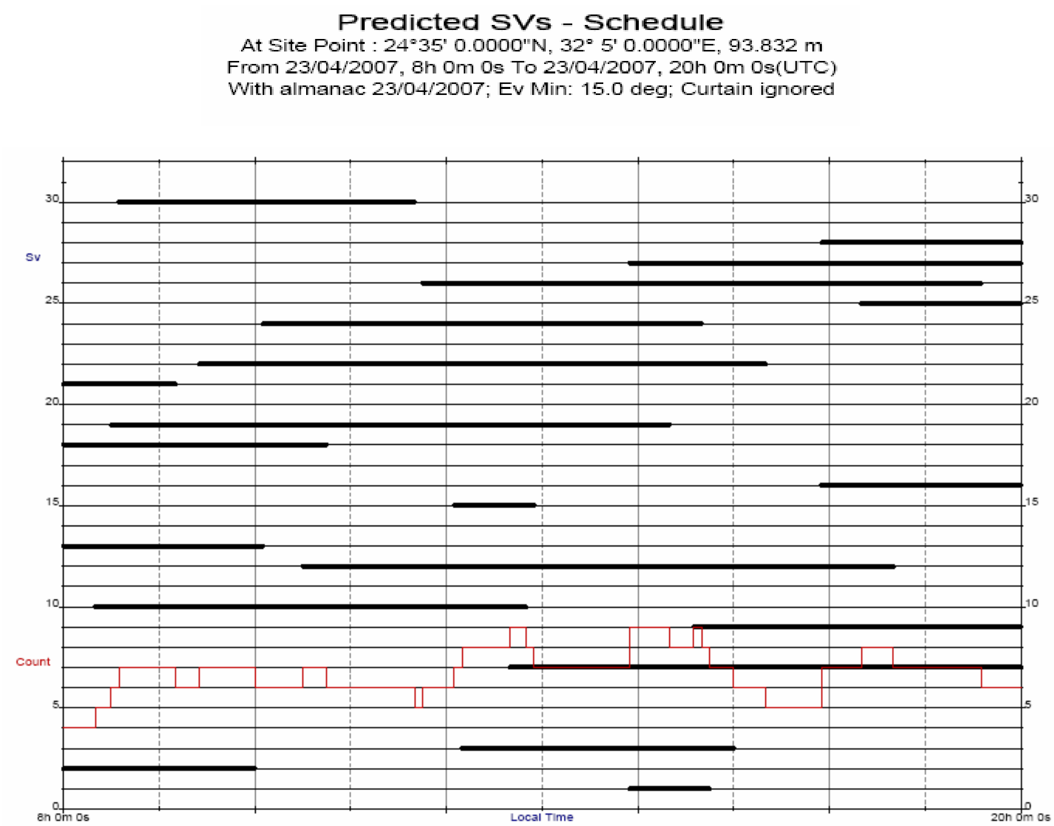
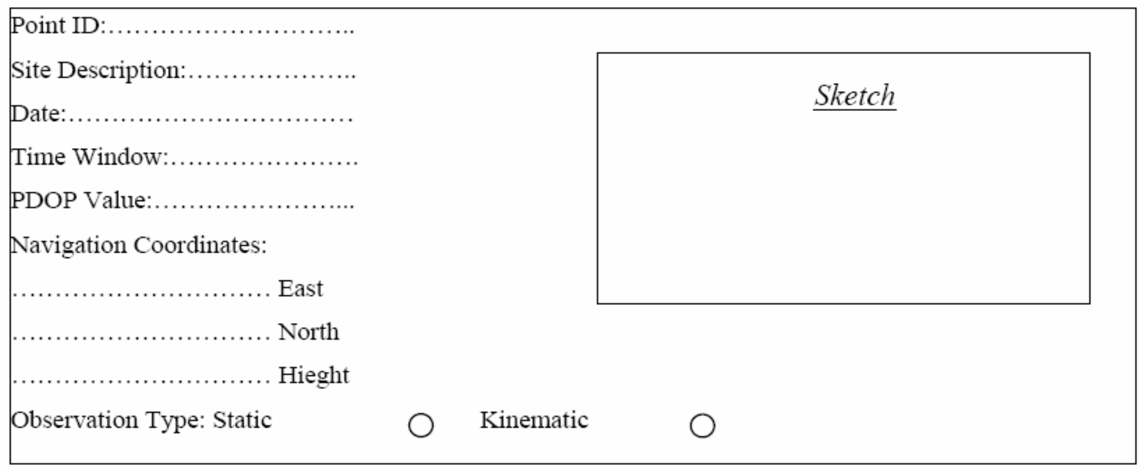


Fig (4-3): Satellite visibility of elevation angle of 15º for 23-4-2007.

4.1.3 Field Equipment

The field equipment includes all instruments that will be used on the field receivers units, tribrachs, tripods and batteries. The receiver type was Promark 3.0 with receivers [Promark 3.0 manual (2005)], GNSS Solution Software [GNSS Solution (2007)] and the Total Station type was TOPCON GTS -712 [TOPCON Manual].

Field sheet is an important to describe the location site, see Figure (4-4)


Point ID:………………………..
Site Description:………………..
Date:……………………………
Time Window:…………………..
PDOP Value:…………………...
Navigation Coordinates:
………………………… East
………………………… North
………………………… Hieght
Observation Type: Static ○ Kinematic ○

Sketch

Fig (4-4): Used Field Sheet.

4.2 Field Work

4.2.1 Static GPS Surveying

The objective of this surveying type is to detect the relative positioning accuracy of GPS Static surveying [2.8.2.1] using single frequency receivers (Promark 3.0).

To study the relative accuracy of the GPS Static Technique, there are some parameters affecting the observation techniques and processing of data such as elevation angle, sample rate, baseline length, observation time, multipath effect and the processing software [(Eckl et al, 2001)], so to get good accuracy assessment of the study one must fix all parameters and change one parameter. So the studied parameters in this research were arranged as given in Table (4-1)

Table (4-1): Scheme of Field Work.

	Group	Test Type	Purpose
1	Group 1	Change the elapsed time for every baseline length	To detect the best elapsed time for every baseline length
2		Change baseline length up to 20 km (same atmospheric conditions)	To detect the relative accuracy of every baseline length
3		Change the used processing ephemeris from Broadcast ephemeris to final IGS ephemeris.	To detect the accuracy of using Broadcast ephemeris that sent with the raw data or waiting for 13 days to get the IGS final ephemeris
4		Compare point's coordinates with stand alone surveying technique	To detect the accuracy of using differential GPS Static technique against stand alone technique
5	Group 2	Change baseline length to 31 km (different atmospheric conditions)	To test using single frequency receiver in observed a baseline longer than 20 km
6		Change the elapsed time	To detect the best elapsed time
7		Change the used processing ephemeris from Broadcast ephemeris to final IGS precise ephemeris.	To detect the accuracy of using Broad cast ephemeris that sent with the raw data or waiting for 13 days to get the IGS precise ephemeris
8		Compare point's coordinates with stand alone surveying technique	To detect the accuracy of using differential GPS Static technique against stand alone technique

4.2.1.1 Group 1

Group 1 includes surveying different baselines length up to 20 km; each baseline had different elapsed time intervals and processes them using broadcast Ephemeris and Precise final IGS Ephemeris. The research studies the effect of change baselines length and changes the elapsed time for every baseline to detect the best elapsed time for every baseline and the relative vector error (ppm) under certain conditions of PDOP values and visible Satellites numbers.

- Field Location

This group of field work was done on Aswan –Edfo Desert Road for the all baselines as shown on Figure (4-5), where this location had the different basic location considerations.

- Methodology

Using of two single frequency receivers (Promark 3.0) in all field work, all work done during three days from 19 to 21-4-2007 with the following steps:

- Reconnaissance the field and select the suitable points on the road far away the transportation roadway to protect points and the surveyors.
- At the first day, record the navigation coordinates of points to have the suitable baseline lengths and fixing the points with steel bars to safe points.
- Using the almanac data file to choose the suitable observation time.
- At the second and third day, start surveying the baselines, set up the base point receiver and set up the rover receiver for the observed points one by one.

Fig (4-5): Field Location Map.

- Elapsed time

The elapsed time for all surveyed baselines start with 10 minutes as a minimum time [El-Shazly A., Abdel-Maguid R. (2004)] and increasing this time according to the baseline length as shown on the following Table (4-2).

Note for point 1 and 2; the baselines lengths are small so the elapsed time is up to 20 min. only. For point 7 (the shadowed cells), there was a problem in collected data for elapsed time of 15, 20, 30 minutes (small elapsed time with long length), so we didn't resurvey that elapsed time again. The elapsed times tested for each baseline is shown in Table (4-2).

Table (4-2): Baselines and their Elapsed Times.

Baseline	Baseline Length in meters	Elapsed Time in minutes						
		10	15	20	30	45	60	75
Base point-point 1	2378	√	√	√	_	_	_	_
Base point-point 2	4050	√	√	√	_	_	_	_
Base point-point 3	5624	√	√	√	√	_	_	_
Base point-point 4	8714	√	√	√	√	√	_	_
Base point-point 5	12521	√	√	√	√	√	√	_
Base point-point 6	15018	√	√	√	√	√	√	_
Base point-point 7	16247	√	_	_	_	√	√	√
Base point-point 8	18802	√	√	√	√	√	√	√

- Processing of Collected Data

After collecting the observation data, GNSS Solution Software is used to process data using the Broadcast and final IGS precise Ephemeris. WGS-84 coordinate system and Universal Transverse Mercator, Zone 36 North (UTM) as a map projection were used to have the coordinates of surveyed points on metric values as follow:

Where: Ellip.H is Ellipsoidal height of point in meters.

Vector Rel.Error is vector Relative Error (95% confidence) in m.

Sat.No. is Satellite Number.

- Note that all processing data for 95% accuracy confidence.
- Processing data as differential solution using Broadcast ephemeris see Table (4-3)

Table (4-3): Processing data (Differential- Broadcast Ephemeris).

Point ID	Elapse time	Vector Length (L) in m	Coordinates UTM Zone 36 N East in m	North in m	Ellip.H in m	Vector Rel.Error in m (95% Confidence)	Sat. No.	PDOP
Point 1	10 min	2378.884	484305.401	2672407.958	177.727	0.012	5	4.7
	15 min	2378.885	484305.402	2672407.958	177.728	0.012	7	1.9
	20 min	2378.879	484305.396	2672407.966	177.743	0.012	8	1.6
Point 2	10 min	4050.129	482194.867	2671745.268	187.554	0.020	8	2
	15 min	4050.129	482194.866	2671745.268	187.555	0.020	8	2.1
	20 min	4050.130	482194.866	2671745.267	187.555	0.020	7	2.1
Point 3	10 min	5624.261	479958.626	2677232.635	191.792	0.028	8	1.9
	15 min	5624.261	479958.625	2677232.635	191.786	0.027	8	2
	20 min	5624.261	479958.626	2677232.637	191.788	0.027	8	2.1
	30 min	5624.26	479958.628	2677232.636	191.793	0.027	8	2
Point 4	10 min	8714.055	478978.336	2681009.941	163.648	0.042	9	2.1
	15 min	8714.054	478978.337	2681009.94	163.648	0.042	9	1.8
	20 min	8714.051	478978.339	2681009.939	163.646	0.042	9	1.9
	30 min	8714.049	478978.341	2681009.938	163.644	0.042	9	1.9
	45 min	8714.050	478978.340	2681009.938	163.644	0.042	8	2
Point 5	10 min	12521.401	478108.151	2685155.978	160.526	0.061	8	1.9
	15 min	12521.403	478108.152	2685155.98	160.529	0.061	8	2.1
	20 min	12521.403	478108.152	2685155.981	160.530	0.061	8	1.9
	30 min	12521.400	478108.154	2685155.978	160.526	0.061	8	2
	45 min	12521.398	478108.156	2685155.977	160.527	0.061	8	2
	60 min	12521.399	478108.155	2685155.978	160.530	0.061	6	2.7
Point 6	10 min	15018.770	477616.109	2687776.671	148.458	0.073	7	2.3
	15 min	15018.772	477616.109	2687776.673	148.463	0.073	8	1.9
	20 min	15018.777	477616.107	2687776.68	148.47	0.073	8	2.1
	30 min	15018.776	477616.108	2687776.679	148.467	0.073	8	1.9

Point ID	Elapse time	Vector Length (L) in m	Coordinates UTM Zone 36 N East in m	North in m	Ellip.H in m	Vector Rel.Error in m (95% Confidence)	Sat. No.	PDOP
Point 6	45 min	15018.775	477616.111	2687776.678	148.464	0.073	8	2
	60 min	15018.770	477616.112	2687776.673	148.466	0.073	7	2
Point 7	10 min	16247.170	477384.868	2689048.996	143.989	0.079	7	2.3
	45 min	16247.181	477384.869	2689049.010	143.979	0.079	8	2.1
	60 min	16247.177	477384.865	2689049.004	143.991	0.079	8	1.6
	75 min	16247.163	477384.873	2689048.992	143.993	0.079	8	1.7
Point 8	10 min	18802.689	476898.676	2691662.369	170.168	0.091	8	2.6
	15 min	18802.684	476898.676	2691662.375	170.163	0.091	8	2.2
	20 min	18802.689	476898.684	2691662.379	170.169	0.091	8	2.5
	30 min	18802.695	476898.684	2691662.391	170.134	0.091	9	1.9
	45 min	18802.696	476898.679	2691662.382	170.143	0.091	8	1.9
	60 min	18802.686	476898.686	2691662.378	170.177	0.091	8	1.5
	75 min	18802.664	476898.687	2691662.357	170.136	0.091	9	1.6

- Processing data as differential solution using final IGS Precise ephemeris see Table (4-4)

Table (4-4): Processing data (Differential -Precise final Ephemeris).

Point ID	Elapse time	Vector Length (L) in m	Coordinates UTM Zone 36 N East in m	North in m	Ellip.H in m	Vector Rel.Error in m (95% Confidence)	Sat. No.	PDOP
Point 1	10 min	2378.884	484305.401	2672407.958	177.727	0.012	5	4.7
	15 min	2378.885	484305.402	2672407.958	177.728	0.012	7	1.9
	20 min	2378.879	484305.396	2672407.966	177.743	0.012	8	1.6
Point 2	10 min	4050.129	482194.867	2671745.268	187.554	0.020	8	2
	15 min	4050.129	482194.866	2671745.269	187.555	0.020	9	1.9
	20 min	4050.130	482194.866	2671745.267	187.554	0.020	8	1.9
Point 3	10 min	5624.261	479958.626	2677232.636	191.792	0.028	8	1.9
	15 min	5624.262	479958.625	2677232.635	191.786	0.027	8	2
	20 min	5624.261	479958.626	2677232.637	191.787	0.027	9	1.9
	30 min	5624.261	479958.627	2677232.636	191.794	0.027	8	1.9
Point 4	10 min	8714.054	478978.337	2681009.941	163.648	0.042	9	1.6
	15 min	8714.054	478978.336	2681009.94	163.647	0.042	9	1.7
	20 min	8714.051	478978.339	2681009.938	163.646	0.042	9	1.9
	30 min	8714.05	478978.341	2681009.938	163.645	0.042	9	1.9
	45 min	8714.051	478978.339	2681009.938	163.646	0.042	8	1.9

Point ID	Elapse time	Vector Length (L) in m	Coordinates UTM Zone 36 N			Vector Rel.Error in m (95% Confidence)	Sat. No.	PDOP
			East in m	North in m	Ellip.H in m			
Point 5	10 min	12521.402	478108.152	2685155.98	160.528	0.061	8	1.9
	15 min	12521.403	478108.153	2685155.981	160.529	0.061	9	1.6
	20 min	12521.404	478108.152	2685155.982	160.529	0.061	9	1.7
	30 min	12521.400	478108.154	2685155.978	160.523	0.061	9	1.9
	45 min	12521.399	478108.155	2685155.977	160.529	0.061	8	1.9
	60 min	12521.399	478108.153	2685155.977	160.523	0.061	7	2
Point 6	10 min	15018.770	477616.111	2687776.674	148.464	0.073	8	1.8
	15 min	15018.772	477616.110	2687776.675	148.466	0.073	8	1.8
	20 min	15018.777	477616.109	2687776.681	148.470	0.073	9	1.6
	30 min	15018.776	477616.108	2687776.68	148.465	0.073	9	1.7
	45 min	15018.775	477616.110	2687776.679	148.464	0.073	9	1.9
	60 min	15018.770	477616.109	2687776.673	148.459	0.073	8	1.9
Point 7	10 min	16247.169	477384.869	2689048.996	143.987	0.079	8	1.8
	45 min	16247.182	477384.867	2689049.011	143.979	0.079	8	2
	60 min	16247.175	477384.868	2689049.003	143.993	0.079	8	1.5
	75 min	16247.163	477384.873	2689048.993	143.995	0.079	9	1.6
Point 8	10 min	18802.689	476898.675	2691662.374	170.16	0.091	7	2.9
	15 min	18802.69	476898.676	2691662.378	170.161	0.092	7	3.1
	20 min	18802.701	476898.684	2691662.379	170.169	0.092	8	2.1
	30 min	18802.696	476898.684	2691662.392	170.133	0.091	7	2.1
	45 min	18802.688	476898.676	2691662.382	170.134	0.091	9	1.6
	60 min	18802.669	476898.686	2691662.378	170.177	0.091	9	1.9
	75 min	18802.664	476898.687	2691662.358	170.139	0.091	9	1.6

- Coordinates of points in stand alone technique (without any processing) , see Table (4-5)

Table (4-5): Coordinates data (Stand alone).

Point ID	Elapse time	Coordinates UTM Zone 36 N		
		East in m	North in m	Ellip.H in m
Point 1	10 min	484306.045	2672409.298	180.048
	15 min	484305.579	2672409.602	179.160
	20 min	484306.188	2672408.989	179.332

Point ID	Elapse time	Coordinates UTM Zone 36 N		
		East in m	North in m	Ellip.H in m
Point 2	10 min	482195.515	2671746.610	189.551
	15 min	482195.566	2671746.736	189.327
	20 min	482195.260	2671746.777	189.097
Point 3	10 min	479959.125	2677234.121	193.159
	15 min	479959.130	2677234.000	193.437
	20 min	479959.081	2677234.090	193.971
	30 min	479959.240	2677233.982	194.018
Point 4	10 min	478979.099	2681011.337	165.794
	15 min	478978.898	2681011.351	165.726
	20 min	478978.704	2681011.318	165.723
	30 min	478978.831	2681011.303	165.818
	45 min	478978.878	2681011.377	165.660
Point 5	10 min	478108.972	2685157.278	162.365
	15 min	478108.813	2685157.383	162.489
	20 min	478108.687	2685157.385	162.425
	30 min	478108.741	2685157.388	162.462
	45 min	478108.787	2685157.287	162.566
	60 min	478108.685	2685157.415	162.413
Point 6	10 min	477616.856	2687777.964	150.660
	15 min	477616.800	2687778.069	150.589
	20 min	477616.597	2687778.084	150.630
	30 min	477616.661	2687778.085	150.564
	45 min	477616.853	2687778.074	150.622
	60 min	477616.488	2687778.097	150.400
Point 7	10 min	477384.769	2689047.482	148.090
	45 min	477385.882	2689048.768	146.046
	60 min	477386.365	2689049.631	142.700
	75 min	477386.427	2689049.385	142.459
Point 8	10 min	476899.069	2691661.639	173.935
	15 min	476899.804	2691661.657	174.679
	20 min	476900.079	2691661.660	174.534
	30 min	476900.864	2691662.289	172.685
	45 min	476900.794	2691663.793	169.368
	60 min	476901.494	2691664.173	168.043
	75 min	476900.600	2691662.381	169.026

- Analysis of Processed Data

To evaluate the accuracy of GPS Static Surveying technique; for observed baseline up to 20 km the longest elapsed time was taken as a reference to compute the differences between coordinates (East, North and Ellipsoidal Height) and vectors values [El-Shazly A., Abdel-Maguid R. (2004)], For example the reference time of baseline 12.52 km is 60 minutes, and the relative vector error is equals to 0.061m (5 PPM (5 mm per 1 km)), see Figure (4-6). This conclusion under conditions of (5 to 9) visible satellites and good PDOP values (1.50 to 4.70)for all observation time intervals and after make sure that all data are free from any discrepancies, as shown in Table (4-6), the differences values (δ') equal to the next equation

$$\delta' = \text{Reference value} - \text{Another elapsed time value} \qquad (4\text{-}1)$$

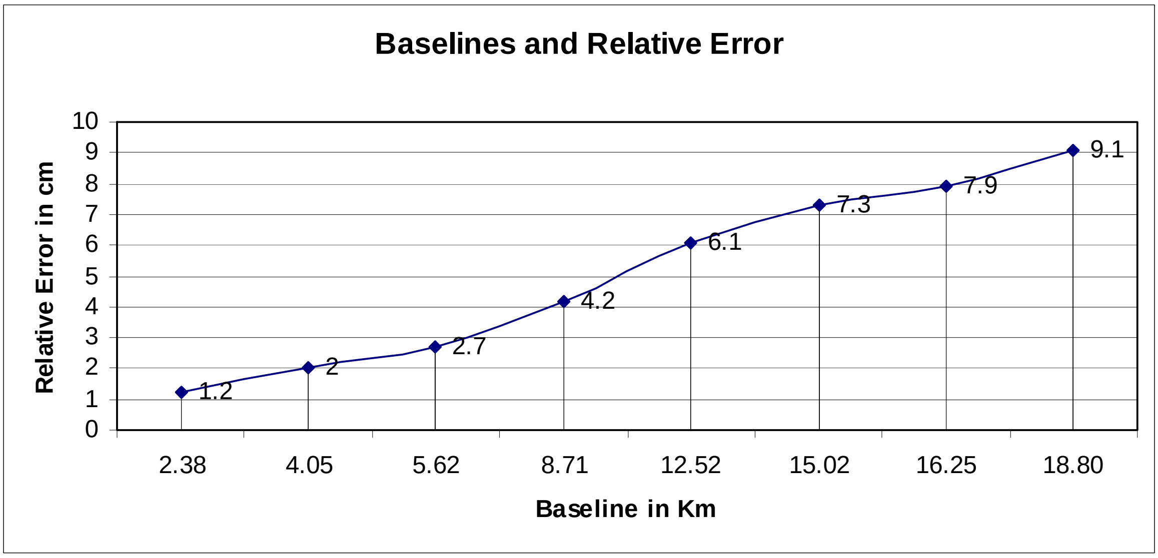


Fig: (4-6) Relation between Baseline length and Relative Error.

Table (4-6): Differences of the Broadcast processing data.

Baseline Length	Elapsed Time	δ'East in m	δ'North in m	δ'H in m	δ'L in m
2.37 km	10 min	-0.005	0.008	0.016	-0.005
	15 min	-0.006	0.008	0.015	-0.006
	20 min	0	0	0	0
4.05 km	10 min	-0.001	-0.001	0.001	0.001
	15 min	0	-0.001	0	0.001
	20 min	0	0	0	0
5.62 km	10 min	-0.001	0.002	0.001	0.001
	15 min	-0.001	0.003	0.001	0.007
	20 min	-0.001	0.002	-0.001	0.005
	30 min	0	0	0	0
8.71 km	10 min	0.004	-0.003	-0.004	-0.005
	15 min	0.003	-0.002	-0.004	-0.004
	20 min	0.001	-0.001	-0.002	-0.001
	30 min	-0.001	0	0	0.001
	45 min	0	0	0	0
12.52 km	10 min	-0.002	0.004	0	0.004
	15 min	-0.004	0.003	-0.002	0.001
	20 min	-0.004	0.003	-0.003	0
	30 min	-0.001	0.001	0	0.004
	45 min	0.001	-0.001	0.001	0.003
	60 min	0	0	0	0
15.02 km	10 min	0.003	0.002	0.008	0.001
	15 min	0.003	0	0.003	-0.002
	20 min	0.005	-0.007	-0.004	-0.008
	30 min	0.004	-0.006	-0.001	-0.006
	45 min	0.001	-0.005	0.002	-0.005
	60 min	0	0	0	0
16.25 km	10 min	0.005	-0.004	0.004	-0.007
	45 min	0.004	-0.018	0.014	-0.018
	60 min	0.008	-0.012	0.002	-0.014
	75 min	0	0	0	0
18.80 km	10 min	0.011	-0.012	-0.032	-0.025
	15 min	0.011	-0.018	-0.027	-0.02
	20 min	0.003	-0.022	-0.033	-0.025
	30 min	0.003	-0.034	0.002	-0.031
	45 min	0.008	-0.025	-0.007	-0.032
	60 min	0.001	-0.021	-0.041	-0.022
	75 min	0	0	0	0

From Table (4-6); the processing results of data by using broadcast ephemeris as follow

- For baseline 2.37 km; δ' East value is smaller than δ' North value and smaller than δ' H values.
- For baseline 4.05 km; the elapsed time of 15 min. is suitable for this baseline where increasing time don't increase the relative accuracy.
- For baselines from 5 km to 10 km; increasing elapsed time of 10 min. increase the relative values of δ' East, δ' North, δ' H and δ' L.
- For baseline from 10 km to 20 km; increasing the elapsed time about the optimum value doesn't increase the relative accuracy.

For the relation between the Broadcast ephemeris processing data, precise final ephemeris data for the points, take the largest observation time for precise final ephemeris as a reference data to detect the relative accuracy as shown in Table (4-7)

Table (4-7): Differences from Precise Ephemeris.

Point ID	Processing Type	Elapsed Time	Coordinates UTM Zone 36 N			
			δ'East in m	δ'North in m	δ'H in m	δ'L in m
2.37 km	Differential Broadcast	10 min	-0.005	0.008	0.016	-0.005
		15 min	-0.006	0.008	0.015	-0.006
		20 min	0	0	0	0
	Differential Precise final	10 min	-0.005	0.008	0.016	-0.005
		15 min	-0.006	0.008	0.015	-0.006
		20 min	0	0	0	0
4.05 km	Differential Broadcast	10 min	-0.001	-0.001	0	0.001
		15 min	0	-0.001	-0.001	0.001
		20 min	0	0	-0.001	0
	Differential Precise final	10 min	-0.001	-0.001	0	0.001
		15 min	0	-0.002	-0.001	0.001
		20 min	0	0	0	0

Point ID	Processing Type	Elapsed Time	Coordinates UTM Zone 36 N			
			δ'East in m	δ'North in m	δ'H in m	δ'L in m
5.62 km	Differential Broadcast	10 min	0.001	0.001	0.002	0
		15 min	0.002	0.001	0.008	0
		20 min	0.001	-0.001	0.006	0
		30 min	-0.001	0	0.001	0.001
	Differential Precise final	10 min	0.001	0	0.002	0
		15 min	0.002	0.001	0.008	-0.001
		20 min	0.001	-0.001	0.007	0
		30 min	0	0	0	0
8.71 km	Differential Broadcast	10 min	0.003	-0.003	-0.002	-0.004
		15 min	0.002	-0.002	-0.002	-0.003
		20 min	0	-0.001	0	0
		30 min	-0.002	0	0.002	0.002
		45 min	-0.001	0	0.002	0.001
	Differential Precise final	10 min	0.002	-0.003	-0.002	-0.003
		15 min	0.003	-0.002	-0.001	-0.003
		20 min	0	0	0	0
		30 min	-0.002	0	0.001	0.001
		45 min	0	0	0	0
12.52 km	Differential Broadcast	10 min	0.002	-0.001	-0.003	-0.002
		15 min	0.001	-0.003	-0.006	-0.004
		20 min	0.001	-0.004	-0.007	-0.004
		30 min	-0.001	-0.001	-0.003	-0.001
		45 min	-0.003	0	-0.004	0.001
		60 min	-0.002	-0.001	-0.007	0
	Differential Precise final	10 min	0.001	-0.003	-0.005	-0.003
		15 min	0	-0.004	-0.006	-0.004
		20 min	0.001	-0.005	-0.006	-0.005
		30 min	-0.001	-0.001	0	-0.001
		45 min	-0.002	0	-0.006	0
		60 min	0	0	0	0
15.02 km	Differential Broadcast	10 min	0	0.002	0.001	0.002
		15 min	0	0	-0.004	-0.001
		20 min	0.002	-0.007	-0.011	-0.007
		30 min	0.001	-0.006	-0.008	-0.005
		45 min	-0.002	-0.005	-0.005	-0.004
		60 min	-0.003	0	-0.007	0.001

Point ID	Processing Type	Elapsed Time	Coordinates UTM Zone 36 N			
			δ'East in m	δ'North in m	δ'H in m	δ'L in m
15.02 km	Differential Precise final	10 min	-0.002	-0.001	-0.005	0
		15 min	-0.001	-0.002	-0.007	-0.002
		20 min	0	-0.008	-0.011	-0.007
		30 min	0.001	-0.007	-0.006	-0.006
		45 min	-0.001	-0.006	-0.005	-0.005
		60 min	0	0	0	0
16.25 km	Differential Broadcast	10 min	0.005	-0.003	0.006	-0.007
		45 min	0.004	-0.017	0.016	-0.018
		60 min	0.008	-0.011	0.004	-0.014
		75 min	0	0.001	0.002	0
	Differential Precise final	10 min	0.004	-0.003	0.008	-0.006
		45 min	0.006	-0.018	0.016	-0.019
		60 min	0.005	-0.01	0.002	-0.012
		75 min	0	0	0	0
18.8 km	Differential Broadcast	10 min	0.011	-0.011	-0.029	-0.025
		15 min	0.011	-0.017	-0.024	-0.02
		20 min	0.003	-0.021	-0.03	-0.025
		30 min	0.003	-0.033	0.005	-0.031
		45 min	0.008	-0.024	-0.004	-0.032
		60 min	0.001	-0.02	-0.038	-0.022
		75 min	0	0.001	0.003	0
	Differential Precise final	10 min	0.012	-0.016	-0.021	-0.025
		15 min	0.011	-0.02	-0.022	-0.026
		20 min	0.003	-0.021	-0.03	-0.037
		30 min	0.003	-0.034	0.006	-0.032
		45 min	0.011	-0.024	0.005	-0.024
		60 min	0.001	-0.02	-0.038	-0.005
		75 min	0	0	0	0

From Table (4-7); the processing results of data by using final precise ephemeris as follow

- For baseline 0 to 5 km; there isn't any change between the two ephemeris.
- For baseline 5.62 km; there is a slight difference for δ' East, δ' North and δ' H where there is no change for δ'L.

- For baseline 10 to 20 km; δ' East value is smaller than δ' North value and smaller than δ' H values. This difference reaches to 11mm in East, 34 mm in North and 38 mm in H as a maximum values.
- For all baselines; the accuracy of maximum time for final precise ephemeris is the same of broadcast one.

Table (4-8) shows stand alone differences where last elapsed time is the reference time as equation (4-1)

Table (4-8): Differences for Stand alone.

Baseline Length	Elapsed Time	δ'East in m	δ'North in m	δ'H in m
2.37 km	10 min	0.143	-0.309	-0.716
	15 min	0.609	-0.613	0.172
	20 min	0.000	0.000	0.000
4.05 km	10 min	-0.255	0.167	-0.454
	15 min	-0.306	0.041	-0.230
	20 min	0.000	0.000	0.000
5.62 km	10 min	0.115	-0.139	0.859
	15 min	0.110	-0.018	0.581
	20 min	0.159	-0.108	0.047
	30 min	0.000	0.000	0.000
8.71 km	10 min	-0.221	0.040	-0.134
	15 min	-0.020	0.026	-0.066
	20 min	0.174	0.059	-0.063
	30 min	0.047	0.074	-0.158
	45 min	0.000	0.000	0.000
12.52 km	10 min	-0.287	0.137	0.048
	15 min	-0.128	0.032	-0.076
	20 min	-0.002	0.030	-0.012
	30 min	-0.056	0.027	-0.049
	45 min	-0.102	0.128	-0.153
	60 min	0.000	0.000	0.000
15.02 km	10 min	-0.368	0.133	-0.260
	15 min	-0.312	0.028	-0.189
	20 min	-0.109	0.013	-0.230
	30 min	-0.173	0.012	-0.164
	45 min	-0.365	0.023	-0.222
	60 min	0.000	0.000	0.000

Baseline Length	Elapsed Time	δ'East in m	δ'North in m	δ'H in m
16.25 km	10 min	1.658	1.903	-5.631
	45 min	0.545	0.617	-3.587
	60 min	0.062	-0.246	-0.241
	75 min	0.000	0.000	0.000
18.80 km	10 min	1.531	0.742	-4.909
	15 min	0.796	0.724	-5.653
	20 min	0.521	0.721	-5.508
	30 min	-0.264	0.092	-3.659
	45 min	-0.194	-1.412	-0.342
	60 min	-0.894	-1.792	0.983
	75 min	0.000	0.000	0.000

From Table (4-8); the processing results of stand alone data as follow

- For baseline points up to 10 km; increasing the elapsed time doesn't increase the accuracy where there is no sequence of difference change according to increasing time.
- For baseline points from 10 to 20 km; increasing the elapsed time increases the accuracy of coordinates.

4.2.1.2 Group 2

This group aims to survey a baseline length of 31 km and process this baseline using Broadcast Ephemeris and Precise final Ephemeris; this group is the same as group 1 but the different of group 2 that to test of using single frequency receiver in observed a baseline larger than 20 km (different atmosphere conditions). [Hoffmann-Wellenhof (2000)]

This baseline was extended from Aswan Faculty of Engineering as a base point to Daraw City as a Rover point at 24-11-2007. The objective of this group is to detect the vector relative error in using the single frequency receiver, compare the two types of Ephemeris (Broadcast and Precise final

IGS), detect the possibility of using single receivers on baselines larger than 20 km and compare all data with stand alone coordinates. The observed data as follow in Table (4-9)

Table (4-9): Processing data of Group 2.

Type	Elapsed Time	Vector Length (L) in m	Coordinates UTM Zone 36 N East in m	North In m	Ellip.H in m	Vector Rel.Error in m (95% Confidence)	Sat. No.	PDOP
Stand alone	15 min	-	492727.100	2699349.000	102.460	-	-	-
	30 min	-	492724.333	2699347.433	106.885	-	-	-
	60 min	-	492723.326	2699346.401	103.220	-	-	-
	90 min	-	492722.619	2699346.309	103.501	-	-	-
	120 min	-	492725.617	2699346.876	99.828	-	-	-
Broadcast	15 min	31068.804	492724.344	2699347.147	101.116	0.297	8.0	2.0
	30 min	31068.862	492724.496	2699347.191	101.263	0.150	9.0	1.9
	60 min	31068.920	492724.486	2699347.250	101.336	0.150	10.0	1.4
	90 min	31068.865	492724.495	2699347.194	101.524	0.150	9	1.8
	120 min	31068.865	492724.449	2699347.199	101.297	0.150	11.0	1.4
Precise final IGS	15 min	31068.800	492724.340	2699347.143	101.113	0.297	8.0	2.0
	30 min	31068.865	492724.493	2699347.195	101.268	0.150	9.0	1.9
	60 min	31068.918	492724.484	2699347.248	101.343	0.150	10.0	1.4
	90 min	31068.864	492724.495	2699347.194	101.524	0.150	9	1.8
	120 min	31068.864	492724.446	2699347.198	101.296	0.150	11.0	1.4

- Analysis of Processed Data

For broadcast ephemeris, take the longest elapsed time as a reference time to compute the differences of observed coordinates and baseline as shown below in Table (4-10), the difference (δ') equal to the following equation

δ' =Longest elapsed time value – Another elapsed time value (4-2)

Table (4-10): Differences of broadcast ephemeris.

Baseline	Elapsed Time	δ'East in m	δ'North in m	δ'H in m	δ'L in m	Vector Rel.Error in m (95% Confidence)
31 Km Broadcast Ephemeris	15 min	0.105	0.052	0.181	0.061	0.297
	30 min	-0.047	0.008	0.034	0.003	0.150
	60 min	-0.037	-0.051	-0.039	-0.055	0.150
	90 min	-0.046	0.005	-0.227	0	0.150
	120 min	0	0	0	0	0.150

From Table (4-10); the processing results of using broadcast ephemeris as follow

- The relative vector error (95% Confidence) of 15 min elapsed time is larger than the others.
- Increasing the elapsed time; there is an improvement in δ'East, δ'North and δ'L values.
- For δ'H values; there is an improvement but in elapsed time of 90 min the difference value increased.

For the relation ship between the Broadcast ephemeris processing data, Precise final ephemeris data and Stand alone data for the points, take the largest observation time for Precise final ephemeris as a reference data to detect the relative accuracy as shown in Table (4-11)

Table (4-11): Differences from Precise final Ephemeris.

Baseline	Elapsed Time	δ'East in m	δ'North in m	δ'H in m	δ'L in m
Stand alone	15 min	-2.654	-1.802	-1.164	_
	30 min	0.113	-0.235	-5.589	–
	60 min	1.120	0.797	-1.924	–
	90 min	1.827	0.889	-2.205	–
	120 min	-1.171	0.322	1.468	–
Broadcast	15 min	0.102	0.051	0.180	0.060
	30 min	-0.050	0.007	0.033	0.002
	60 min	-0.040	-0.052	-0.040	-0.056
	90 min	-0.049	0.005	-0.229	-0.001
	120 min	-0.003	-0.001	-0.001	-0.001
Precise final	15 min	0.106	0.055	0.183	0.064
	30 min	-0.047	0.003	0.028	-0.001
	60 min	-0.038	-0.050	-0.047	-0.054
	90 min	-0.049	0.005	-0.229	0.000
	120 min	0.000	0.000	0.000	0.000

From Tcable (4-11); the analysis results show as follow

- For every elapsed time in broadcast and final precise IGS ephemeris; there is no apparent change in values.
- There is good improvement of coordinates in stand alone where the East error is larger than North values and H error is the largest.
- In increasing the elapsed time; there is an improvement in δ'East, δ'North and δ'L values.
- For δ'H values; there is an improvement but in elapsed time of 90 min the difference value increased.

4.2.2 Kinematic GPS Surveying

4.2.2.1 Digital Elevation Model

The objective of this experiment is to evaluate the absolute accuracy of Digital Elevation Model (DEM) (2.12) between Total Station Technique as reference data and GPS techniques. This evaluation was done by computing the difference between Total Station data and GPS data. Cut and fill volumes, girding, and cross sections were calculated from data and were compared.

- Field Location

This work was done in the new city of EL-SADAKA - Aswan Governorate.

This study includes the following analysis steps:

- Compute the statistics of original data for 10*10 m grid.
- Compute the statistics of original data for 20*20 m in order to study the suitability of increase the grid spacing from 10 to 20 m to save time and cost.
- Generate the contour maps for 10*10 m grid and 20*20 m.
- Generate the grid maps for 10*10 m grid and 20*20 m.
- Compute the cut and fill volumes for 10*10 m grid and 20*20 m for the three observation techniques.
- Draw the selected cross sections of the whole area and compute the statistics data of these cross sections.

- Methodology

1. First visiting the location on Tuesday 15 April 2008 and get an approximately network level by Total Station on the hole area to decide the actual selected area and the best grid spacing between grid lines.

2. Survey the area using Total Station TOPCON GTS 712, by using selected positions for the base point and the back side point.
3. Drawing the contour map for the location, the selected area equal 100*100 m and spacing of 10 m in both sides.
4. Locate the grid points in the field using Total Station. Fig (4-7)

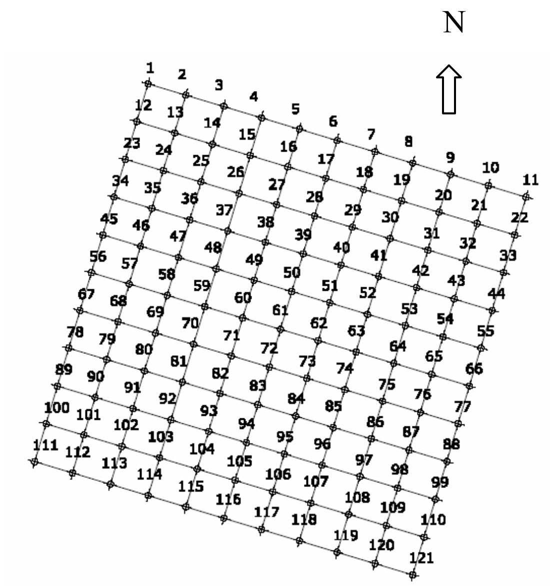


Fig (4-7): Area Grid.

5. two GPS measured points were used as an occupied and back sight points to tie the Total Station coordinates to The WGS 84 system / UTM zone 36 North as map projection, with EGM-96 geoid model, see table (4-12).

Table (4-12) Reference points coordinates.

Point ID	East	North	Orthometric height	Notes
1	492325.780	2665596.952	178.214	-----
11	492421.503	2665568.251	181.396	-----

6. Survey the location area using three techniques:
   6.1. Total Station Technique

By using the two control points, grid height can be surveyed by Total Station for all grid points.

6.2. Stop & Go GPS Technique

In this technique, put the base receiver above point 1 and first initialize the rover receiver by using bar for 5 minutes, this initialization time is an important to ensure that your kinematic surveys, whether continuous or Stop & Go, will reach centimeter-level accuracies through post-processing. The elapsed time one point is 15 seconds with 1 second as an epoch in the base receiver and the rover receiver. Note that the base point and rover point have the same sample rate or the same epoch [Promark 3.0 manual (2005)], satellite numbers are in between (8 - 10) and PDOP value is in between (2 - 3.50) and the GPS relative accuracy of height in between (0.30 to 0.60) cm (95% confidence) [GNSS Solution (2007)].

6.3. Kinematic GPS Technique

This technique start with initialization by bar for rover receiver on base point and moving by surveyor above points with elapsed time on point of 1 second but epoch every 5 seconds (one epoch every 5 seconds), the base point have an epoch every 5 seconds. Note that the base point and rover point have the same sample rate or the same epoch [Promark 3.0 manual (2005)], satellite numbers are in between (11 - 12) and PDOP value is in between (1.6 – 1.7) but the GPS relative accuracy of height in between (3.40 to 10.20) cm (95% confidence) [GNSS Solution (2007)].

Topographic map with actual coordinates is shown in figure (4-8)

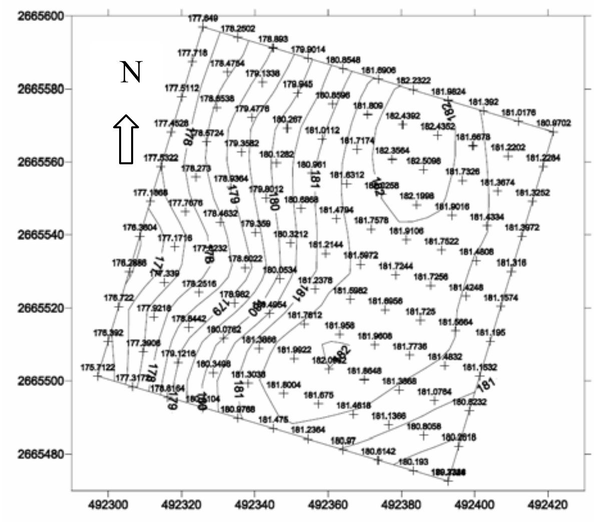


Fig (4-8): Topographic map in actual coordinates.

7. Convert the original data from the actual x and y coordinates to assumed coordinates in the north direction, just to draw the contour maps and compute the cut and fill volumes without any error in creating the grid by Surfer 8.0 Package Program [Golden Software Inc.(2002)], kriging interpolation method [Othman M. Awad (2005)] as shown in figure (4-9).

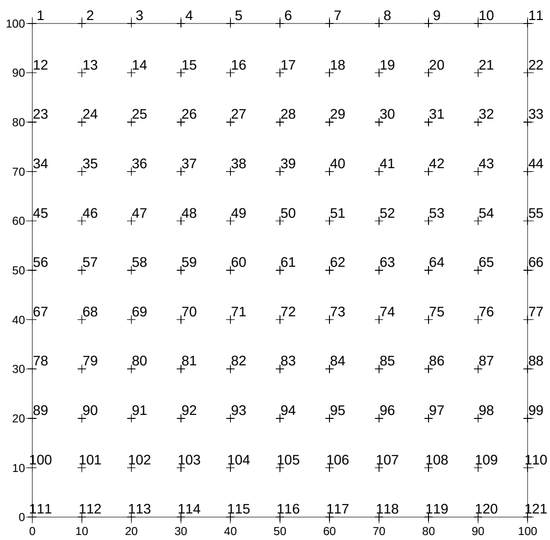


Fig (4-9): New coordinates of the studied area and points ID.

Table (4-13) shows the collected data and the differences between the observations techniques according to the following equations

Diff. 1 = Total Station Height - S&G Height in cm (4-3)

Diff. 2 = Total Station Height – Kinematic Height in cm (4-4)

Table (4-13) Collected data for DEM.

Point ID	X	Y	Total Station H in m	S&G H in m	Kinematic in m	Diffr.1 in cm	Diffr.2 in cm
111	0	0	175.712	175.633	175.655	7.9	5.70
112	10	0	177.317	177.221	177.180	9.6	13.70
113	20	0	178.816	178.727	178.710	8.9	10.60
114	30	0	180.110	180.009	179.991	10.1	11.90
115	40	0	180.977	180.879	180.887	9.8	9.00
116	50	0	181.475	181.361	181.284	11.4	19.10
117	60	0	181.236	181.149	181.121	8.7	11.50
118	70	0	180.970	180.868	180.884	10.2	8.60
119	80	0	180.614	180.512	180.477	10.2	13.70
120	90	0	180.193	180.088	180.080	10.5	11.30
121	100	0	179.735	179.631	179.657	10.4	7.80
100	0	10	176.392	176.310	176.339	8.2	5.30
101	10	10	177.391	177.300	177.507	9.1	-11.60
102	20	10	179.122	179.031	179.073	9.1	4.90
103	30	10	180.35	180.26	180.296	9	5.40
104	40	10	181.304	181.211	181.177	9.3	12.70
105	50	10	181.8	181.69	181.689	11	11.10
106	60	10	181.675	181.593	181.590	8.2	8.50
107	70	10	181.462	181.351	181.378	11.1	8.40
108	80	10	181.137	181.034	180.993	10.3	14.40
109	90	10	180.806	180.712	180.670	9.4	13.60
110	100	10	180.262	180.161	180.105	10.1	15.70
89	0	20	176.722	176.637	176.451	8.5	27.10
90	10	20	177.922	177.833	177.855	8.9	6.70
91	20	20	178.844	178.748	178.738	9.6	10.60
92	30	20	180.076	179.984	179.930	9.2	14.60
93	40	20	181.389	181.284	181.289	10.5	10.00
94	50	20	181.992	181.89	181.891	10.2	10.10
95	60	20	182.096	182.004	181.979	9.2	11.70
96	70	20	181.865	181.75	181.740	11.5	12.50
97	80	20	181.387	181.272	181.292	11.5	9.50
98	90	20	181.076	180.977	180.918	9.9	15.80
99	100	20	180.823	180.712	180.679	11.1	14.40
78	0	30	176.289	176.199	176.242	9	4.70
79	10	30	177.339	177.261	177.477	7.8	-13.80

Point ID	X	Y	Total Station H in m	S&G H in m	Kinematic in m	Diffr.1 in cm	Diffr.2 in cm
80	20	30	178.252	178.16	178.230	9.2	2.20
81	30	30	178.982	178.895	178.847	8.7	13.50
82	40	30	180.495	180.403	180.393	9.2	10.20
83	50	30	181.761	181.655	181.639	10.6	12.20
84	60	30	181.958	181.860	182.006	9.8	-4.80
85	70	30	181.961	181.849	181.821	11.2	14.00
86	80	30	181.774	181.670	181.641	10.4	13.30
87	90	30	181.483	181.367	181.362	11.6	12.10
88	100	30	181.153	181.053	181.019	10	13.40
67	0	40	176.360	176.268	176.26	9.2	10.00
68	10	40	177.172	177.074	177.277	9.8	-10.50
69	20	40	177.823	177.726	177.696	9.7	12.70
70	30	40	178.602	178.494	178.568	10.8	3.40
71	40	40	180.053	179.964	179.888	8.9	16.50
72	50	40	181.238	181.141	181.383	9.7	-14.50
73	60	40	181.598	181.506	181.483	9.2	11.50
74	70	40	181.696	181.614	181.574	8.2	12.20
75	80	40	181.725	181.626	181.586	9.9	13.90
76	90	40	181.566	181.463	181.432	10.3	13.40
77	100	40	181.195	181.097	181.019	9.8	17.60
56	0	50	177.187	177.096	177.149	9.1	3.80
57	10	50	177.768	177.682	177.824	8.6	-5.60
58	20	50	178.463	178.372	178.340	9.1	12.30
59	30	50	179.359	179.264	179.255	9.5	10.40
60	40	50	180.321	180.230	180.208	9.1	11.30
61	50	50	181.214	181.123	181.282	9.1	-6.80
62	60	50	181.597	181.505	181.482	9.2	11.50
63	70	50	181.724	181.628	181.593	9.6	13.10
64	80	50	181.726	181.623	181.674	10.3	5.20
65	90	50	181.425	181.324	181.292	10.1	13.30
66	100	50	181.157	181.047	181.016	11	14.10
45	0	60	177.532	177.434	177.314	9.8	21.80
46	10	60	178.273	178.179	178.299	9.4	-2.60
47	20	60	178.936	178.842	178.831	9.4	10.50
48	30	60	179.801	179.691	179.651	11	15.00
49	40	60	180.687	180.602	180.575	8.5	11.20
50	50	60	181.479	181.388	181.337	9.1	14.20
51	60	60	181.758	181.660	181.679	9.8	7.90
52	70	60	181.911	181.833	181.782	7.8	12.90
53	80	60	181.752	181.653	181.710	9.9	4.20
54	90	60	181.481	181.377	181.359	10.4	12.20
55	100	60	181.316	181.203	181.183	11.3	13.30
34	0	70	177.453	177.349	177.477	10.4	-2.40
35	10	70	178.572	178.493	178.543	7.9	2.90

Point ID	X	Y	Total Station H in m	S&G H in m	Kinematic in m	Diffr.1 in cm	Diffr.2 in cm
36	20	70	179.358	179.255	179.383	10.3	-2.50
37	30	70	180.128	180.023	179.986	10.5	14.20
38	40	70	180.961	180.871	180.962	9	-0.10
39	50	70	181.631	181.537	181.514	9.4	11.70
40	60	70	182.026	181.932	181.765	9.4	26.10
41	70	70	182.200	182.117	182.072	8.3	12.80
42	80	70	181.902	181.811	181.767	9.1	13.50
43	90	70	181.433	181.326	181.286	10.7	14.70
44	100	70	181.397	181.309	181.259	8.8	13.80
23	0	80	177.511	177.409	177.532	10.2	-2.10
24	10	80	178.654	178.557	178.557	9.7	9.70
25	20	80	179.478	179.378	179.411	10	6.70
26	30	80	180.267	180.153	180.134	11.4	13.30
27	40	80	181.011	180.912	180.903	9.9	10.80
28	50	80	181.717	181.622	181.570	9.5	14.70
29	60	80	182.356	182.266	182.252	9	10.40
30	70	80	182.510	182.416	182.315	9.4	19.50
31	80	80	181.733	181.638	181.613	9.5	12.00
32	90	80	181.367	181.259	181.247	10.8	12.00
33	100	80	181.325	181.226	181.176	9.9	14.90
12	0	90	177.718	177.622	177.623	9.6	9.50
13	10	90	178.475	178.382	178.381	9.3	9.40
14	20	90	179.134	179.043	179.165	9.1	-3.10
15	30	90	179.945	179.834	179.779	11.1	16.60
16	40	90	180.860	180.750	180.711	11	14.90
17	50	90	181.809	181.715	181.685	9.4	12.40
18	60	90	182.439	182.332	182.394	10.7	4.50
19	70	90	182.435	182.338	182.281	9.7	15.40
20	80	90	181.668	181.576	181.584	9.2	8.40
21	90	90	181.220	181.117	181.090	10.3	13.00
22	100	90	181.228	181.127	181.084	10.1	14.40
1	0	100	177.649	177.618	177.592	3.1	5.70
2	10	100	178.250	178.158	178.115	9.2	13.50
3	20	100	178.893	178.800	179.003	9.3	-11.00
4	30	100	179.901	179.805	179.787	9.6	11.40
5	40	100	180.855	180.756	180.852	9.9	0.30
6	50	100	181.691	181.584	181.530	10.7	16.10
7	60	100	182.232	182.142	182.113	9	11.90
8	70	100	181.982	181.880	181.817	10.2	16.50
9	80	100	181.392	181.285	181.353	10.7	3.90
10	90	100	181.018	180.910	180.878	10.8	14.00
11	100	100	180.970	180.877	180.832	9.3	13.80

- Analysis of collected data

To compare between the surveying techniques, contour map will be generate the contour maps for the studied area in each technique and generate the selected cross sections for the all techniques as shown in figure (4-10).

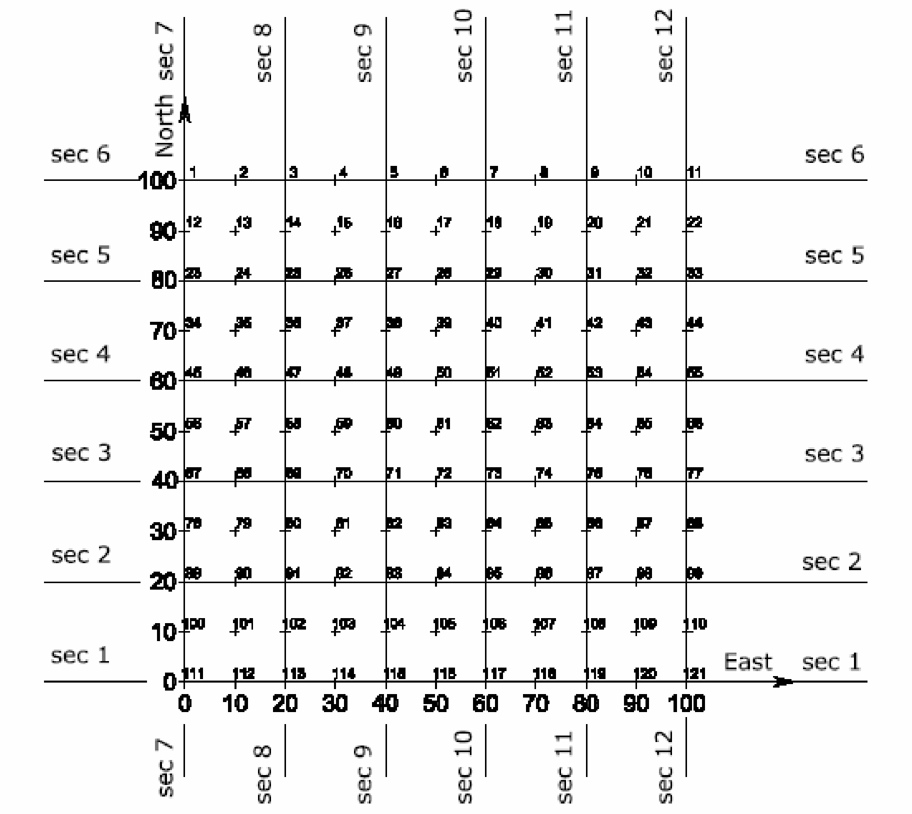


Fig (4-10): Cross sections of the studied area.

DEM creates using three surveying techniques, first by Total Station technique as a reference data, STOP and GO technique, Kinematic technique.

Analysis of collected data includes the following steps:

- Compute the statistics of original data for 10*10 m grid and 20*20 m.
- Drawing the contour maps for 10*10 m grid and 20*20 m.
- Drawing the grid maps for 10*10 m grid and 20*20 m.

- Compute the cut and fill volumes for 10*10 m grid and 20*20 m for the three observation techniques.
- Draw the selected cross sections of the whole area and compute the statistics data of these cross sections.
- **Statistics of original data differences**

The analysis statistics of observed data show in the following table (4-14)

Table (4-14) Statistics of collected data

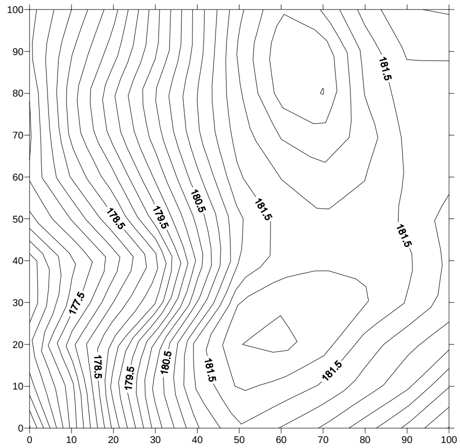
Grid type	Diffr. type	RMSE cm	Mean value cm	Max. value cm	Min. value cm
10*10 m	Diff.1 in cm	9.7	9.6	11.6	3.1
	Diff.2 in cm	12.0	9.5	25.5	-14.5
20*20 m	Diff.1 in cm	9.5	9.4	11.5	3.1
	Diff.2 in cm	12.1	10.2	27.1	-11.0

Table (4-14) shows the following notes

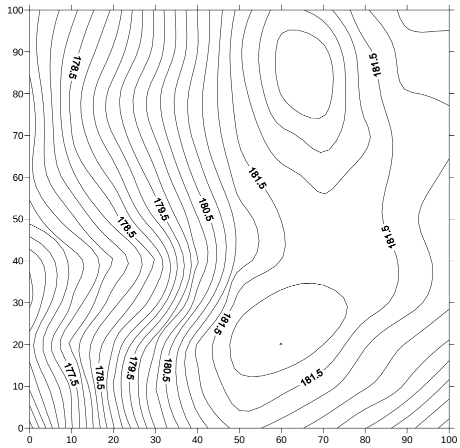
- The statistics data shows the RMSE value of Stop and Go technique is smaller than Kinematic GPS technique for grid 10*10 m and 20*20 m.
- The maximum value of difference for Stop and Go is smaller than kinematic technique for grid 10*10 m and 20*20.

- **Contour Maps for10*10 m and 20*20 m grids**

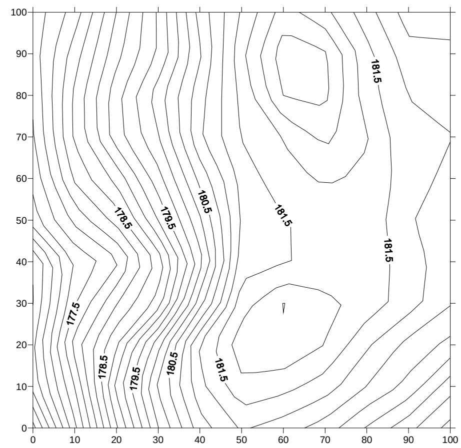
Contour maps are using Surfer 8.0 package as follow in fig. (4-11)



Total Station Technique

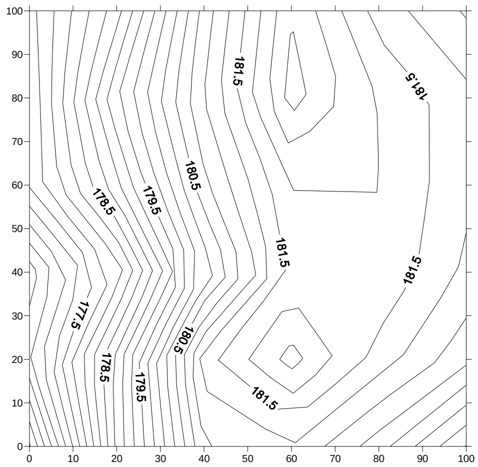


Stop and Go Technique

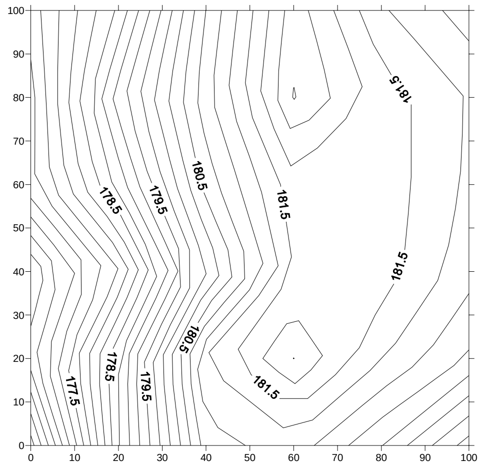


kinematic Technique

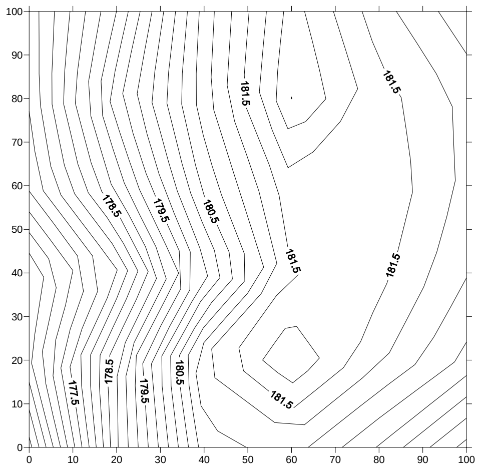
*Fig (4-11): Contour maps for 10*10 m Contour Interval 0.25 m.*



Total Station Technique



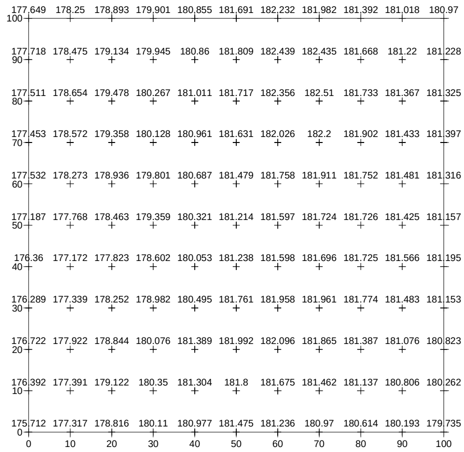
Stop and Go Technique



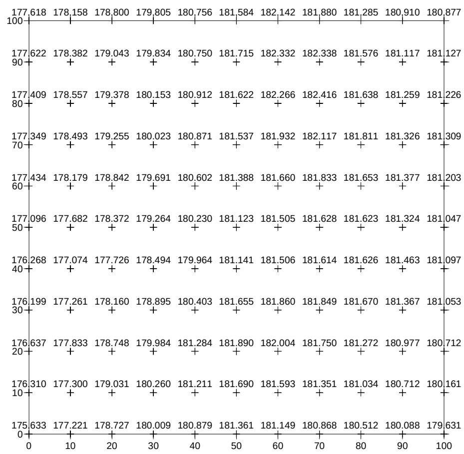
kinematic Technique

*Fig (4-12): Contour maps for 20*20 m Contour Interval 0.25 m.*

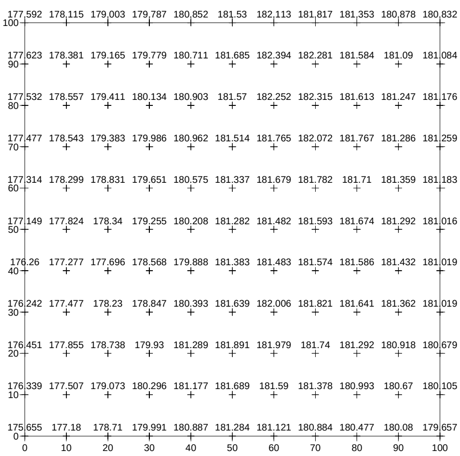
- **Grid Maps for10*10 m and 20*20 m**



Total Station Technique

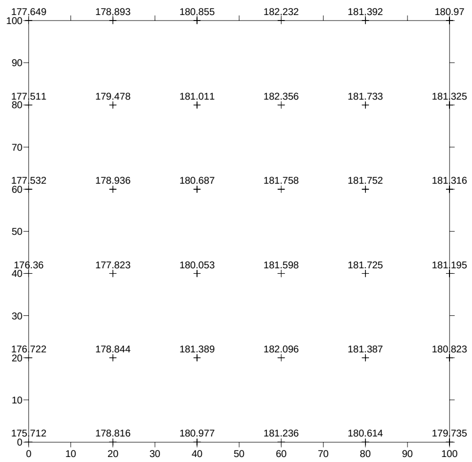


Stop and Go Technique

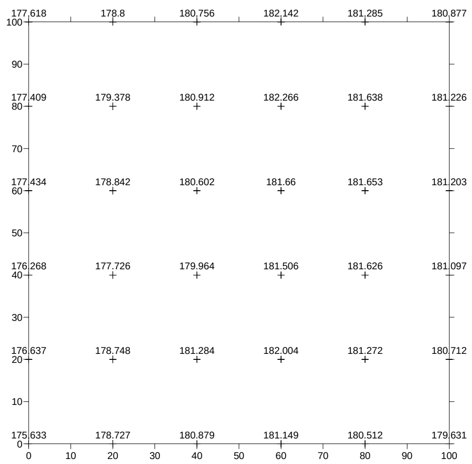


kinematic Technique

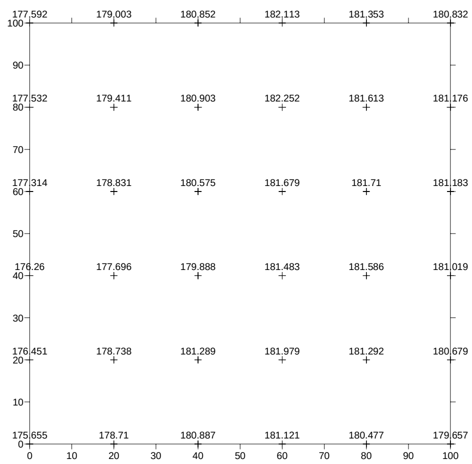
*Fig (4-13): Grid maps for 10*10 m.*



Total Station Technique



Stop and Go Technique



kinematic Technique

*Fig (4-14): Grid maps for 20*20 m.*

- **Cut and Fill Volumes of the Field Area**

To compare between the surveying techniques by using Surfer 8.0 software and Kriging interpolation method, we can compute the cut and fill volumes using a number of grading levels as shown in table (4-15).

Table (4-15) Volumes of Earth Work

Level	Grid	Volumes	Total Station	S&G	Kinematic
181 m	10*10	Cut m^3	3418.438	2894.598	2824.848
		Fill m^3	8800.238	9246.715	9142.548
		Net m^3	-5381.800	-6352.117	-6317.700
	20*20	Cut m^3	2903.089	2403.930	2363.739
		Fill m^3	8862.889	9323.796	9368.740
		Net m^3	-5959.800	-6919.867	-7005.000
180 m	10*10	Cut m^3	9591.806	8924.630	8838.822
		Fill m^3	4973.606	5276.747	5156.522
		Net m^3	4618.200	3647.883	3682.300
	20*20	Cut m^3	9003.689	8343.392	8288.878
		Fill m^3	4963.489	5263.259	5293.878
		Net m^3	4040.200	3080.133	2995.000
179 m	10*10	Cut m^3	16908.985	16141.724	16067.759
		Fill m^3	2290.785	2493.840	2385.459
		Net m^3	14618.200	13647.883	13682.300
	20*20	Cut m^3	16335.654	15575.335	15533.846
		Fill m^3	2295.454	2495.202	2538.847
		Net m^3	14040.200	13080.133	12995.000
178 m	10*10	Cut m^3	25290.877	24420.165	24381.608
		Fill m^3	672.677	772.281	699.308
		Net m^3	24618.200	23647.883	23682.300
	20*20	Cut m^3	24705.248	23847.711	23811.872
		Fill m^3	665.048	767.578	816.872
		Net m^3	24040.200	23080.133	22995.000
177 m	10*10	Cut m^3	34699.892	33753.745	33779.971
		Fill m^3	81.692	105.862	97.671
		Net m^3	34618.200	33647.883	33682.300
	20*20	Cut m^3	34117.854	33181.252	33114.969
		Fill m^3	77.654	101.118	119.969
		Net m^3	34040.200	33080.133	32995.000

The percentage of the difference in earthwork (cut and fill) is as follow

$$\Delta cf = \left|\left| \frac{cf1 - cf2}{cf1} \right|\right| \quad \%$$

Where

Δcf: the percentage difference in earthwork volume.

cf1: the net volume for total station survey.

cf2: the net volume for the others techniques. [Othman M. Awad (2005)]

See table (4-16)

Table (4-16) Percentage of Earthwork.

Level	Grid	Total Station	Stop and Go	kinematic
181 m	10 m	0.000 %	18.030 %	17.390 %
	20 m	10.740 %	28.579 %	30.161 %
180 m	10 m	0.000 %	21.011 %	20.265 %
	20 m	12.516 %	33.304 %	35.148 %
179 m	10 m	0.000 %	6.638 %	6.402 %
	20 m	3.954 %	10.522 %	11.104 %
178 m	10 m	0.000 %	3.941 %	3.802 %
	20 m	2.348 %	6.248 %	6.593 %
177 m	10 m	0.000 %	2.803 %	2.703
	20 m	1.670 %	4.443 %	4.689

Form table (4-16);

- From percentage of the cut and fill volumes; the arrangement of observation techniques as follow Total Station (10 m), Total Station (20 m), Kinematic (10 m), Stop and Go (10m), Stop and Go (20m) and kinematic (20 m).
-

- **<u>Cross Sections of the Field Area</u>**
  - East and West Cross Sections

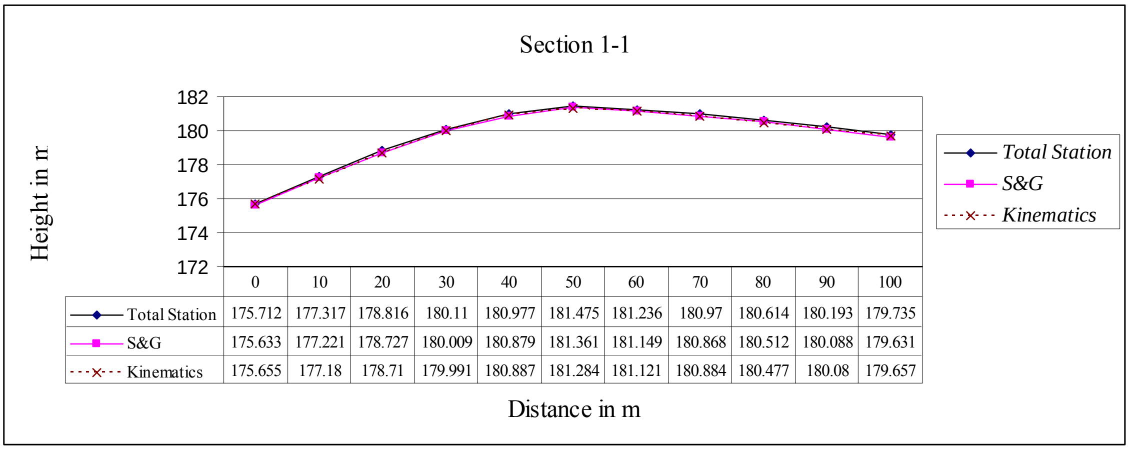


Fig (4-15): Cross Section 1-1

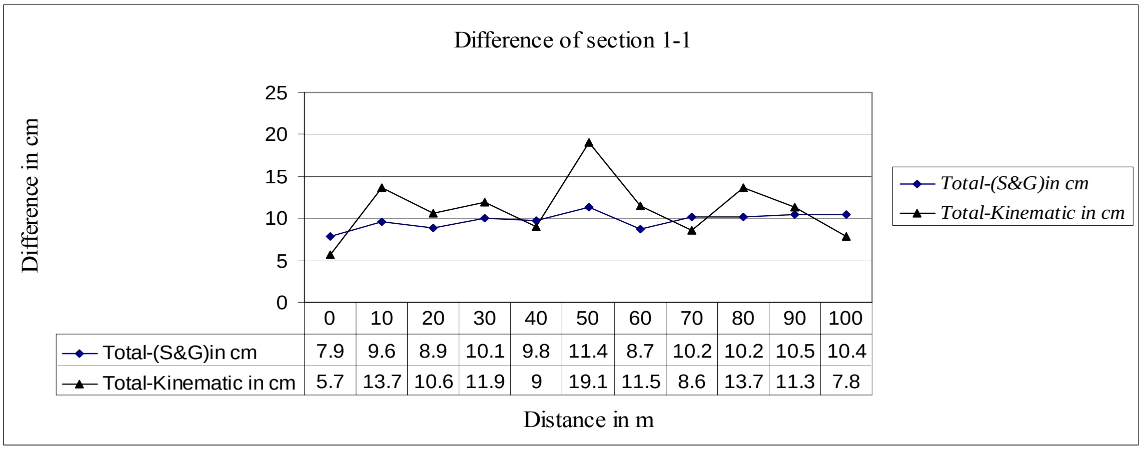


Fig (4-16): Difference of Section 1-1

Table (4-17): Statistics of Cross Section 1-1

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.8	9.7	11.4	7.9
Diff.2 in cm	11.6	12.2	19.1	5.7

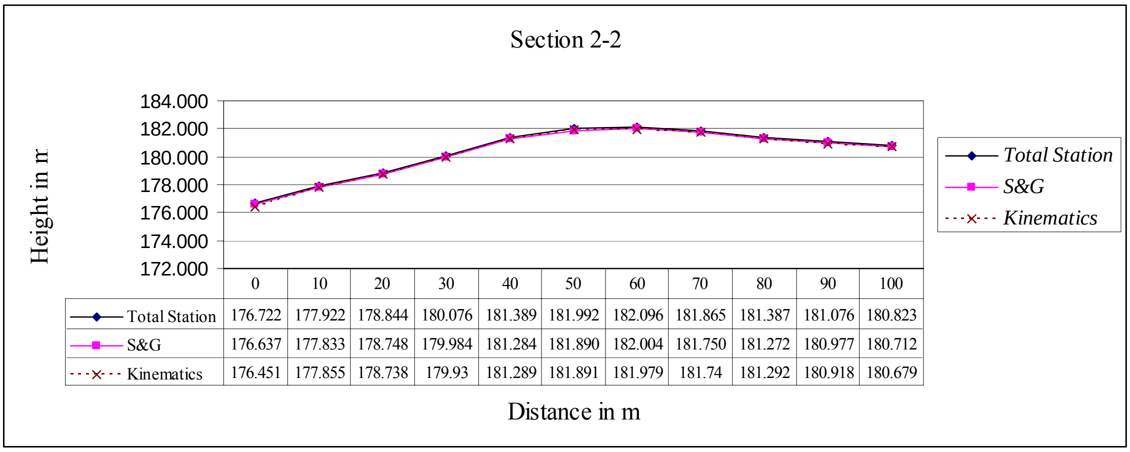


Fig (4-17): Cross Section 2-2

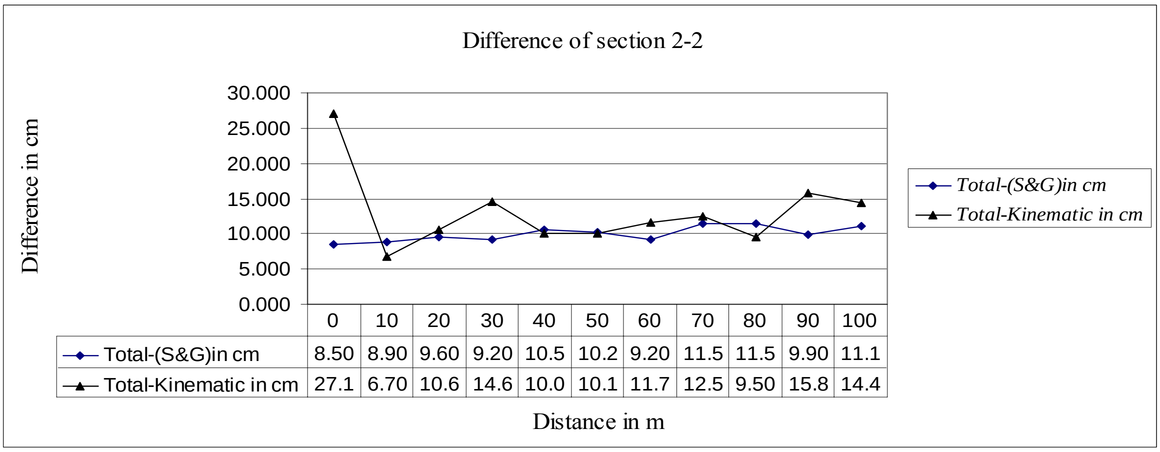


Fig (4-18): Difference of Section 2-2

Table (4-18): Statistics of Cross Section 2-2

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	10.0	10.0	11.5	8.5
Diff.2 in cm	13.9	13.0	27.1	6.7

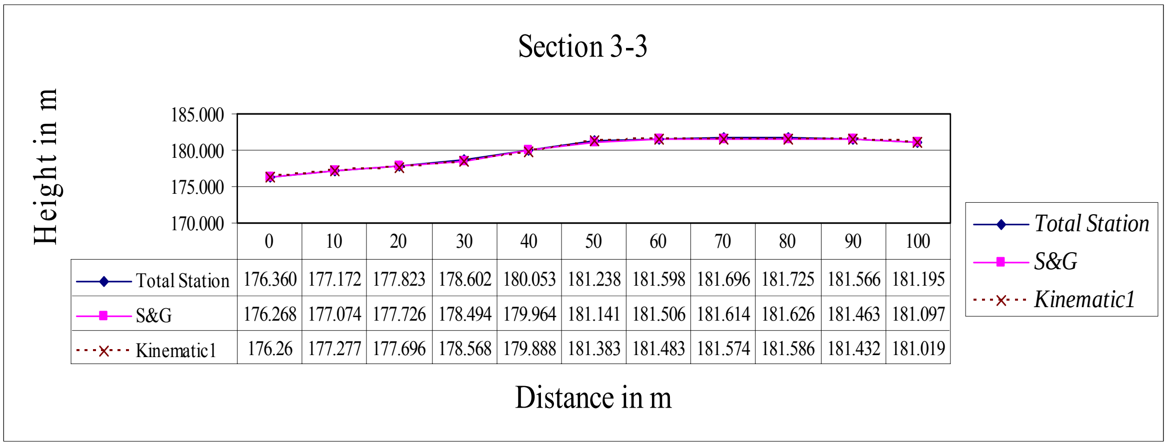


Fig (4-19): Cross Section 3-3

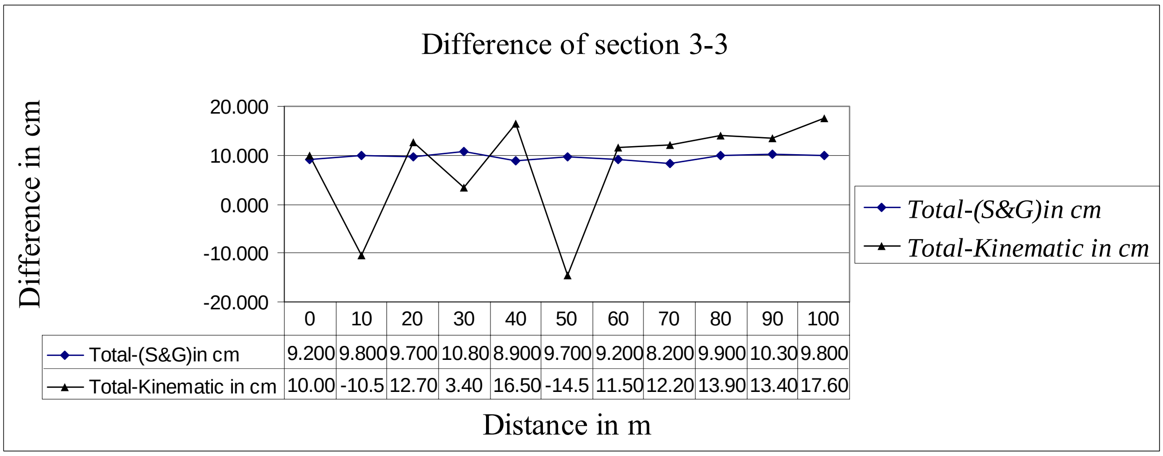


Fig (4-20): Difference of Section 3-3

Table (4-19): Statistics of Cross Section 3-3

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.6	9.5	10.8	8.2
Diff.2 in cm	12.8	7.8	17.6	-14.5

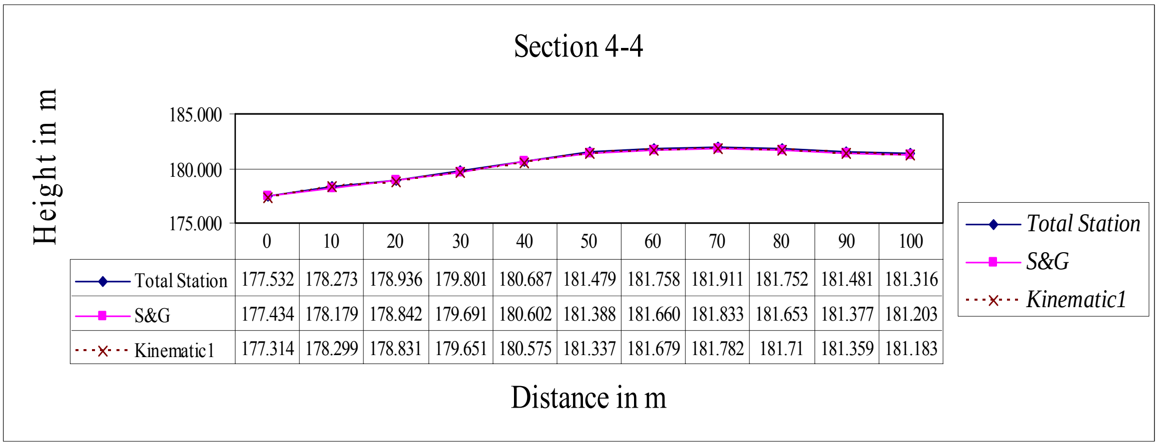


Fig (4-21): Cross Section 4-4

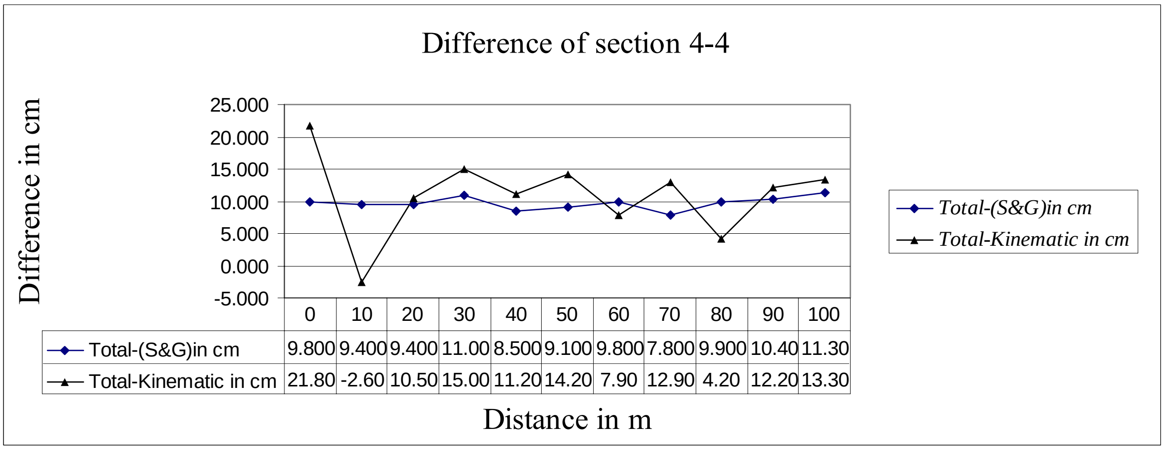


Fig (4-22): Difference of Section 4-4

Table (4-20): Statistics of Cross Section 4-4

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.7	9.6	11.3	7.8
Diff.2 in cm	12.5	10.9	21.8	-2.6

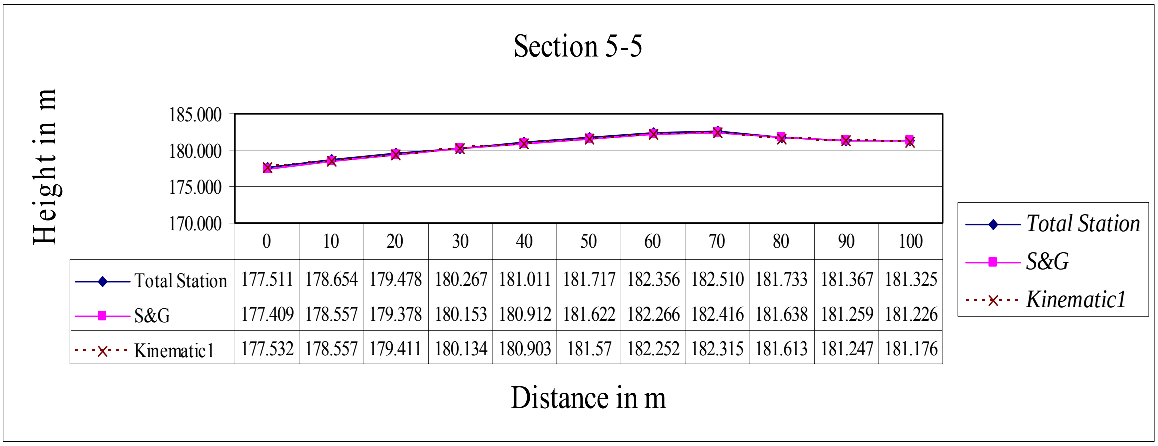


Fig (4-23): Cross Section 5-5

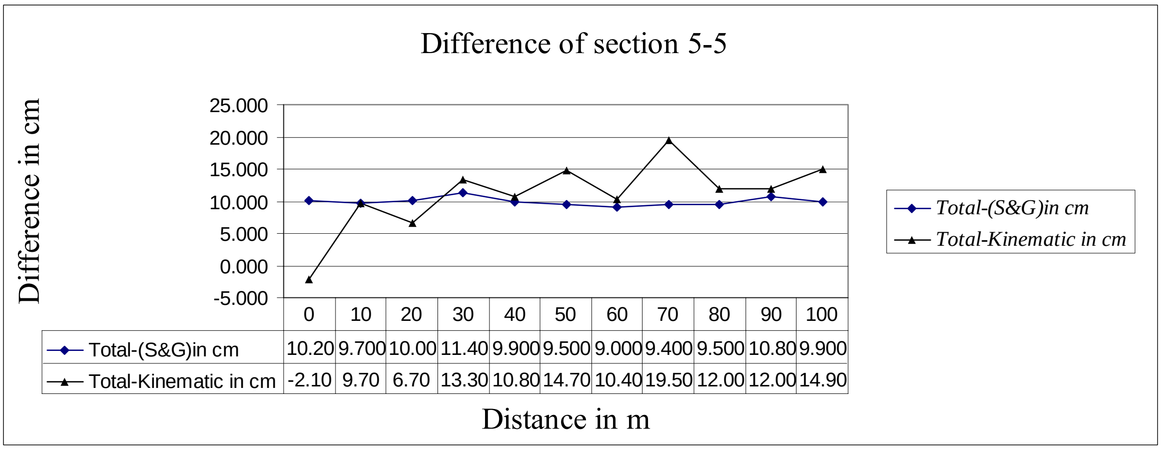


Fig (4-24): Difference of Section 5-5

Table (4-21): Statistics of Cross Section 5-5

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.9	9.9	11.4	9.0
Diff.2 in cm	12.2	11.0	19.5	-2.1

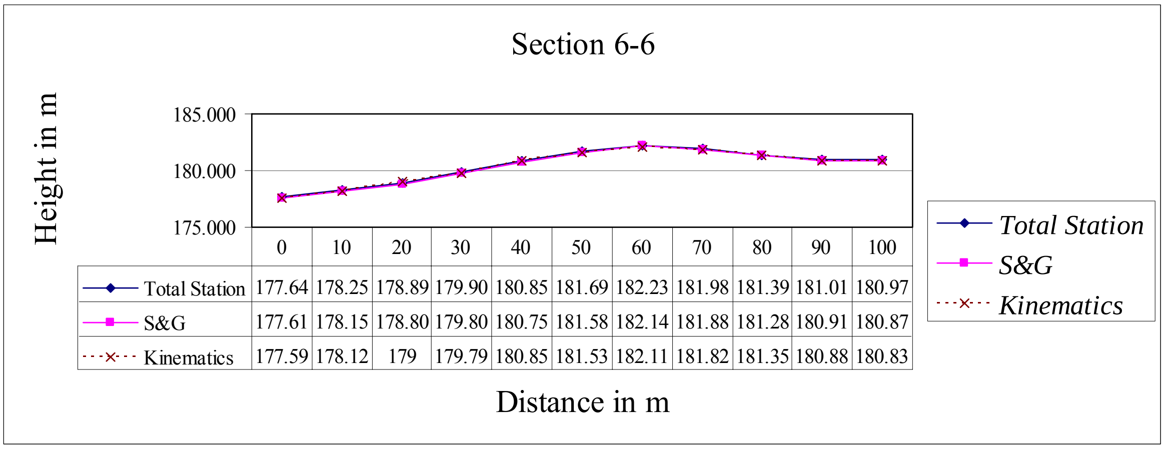


Fig (4-25): Cross Section 6-6

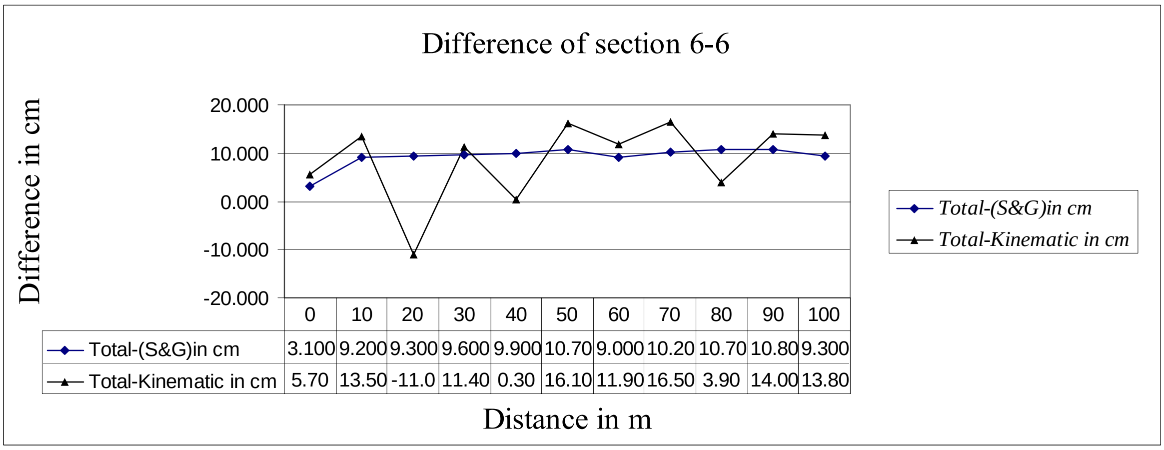


Fig (4-26): Difference of Section 6-6

Table (4-22): Statistics of Cross Section 6-6

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.5	9.2	10.8	3.1
Diff.2 in cm	11.8	8.7	16.1	-11.0

Form the East – west Cross Sections

- RMSE for kinematic difference is greater than Stop and Go values.
- RMSE values for Stop and Go are in between (9.5 cm to 10.0 cm) but for kinematic are in between (11.6 cm to 13.9 cm).

- North – South Cross Sections

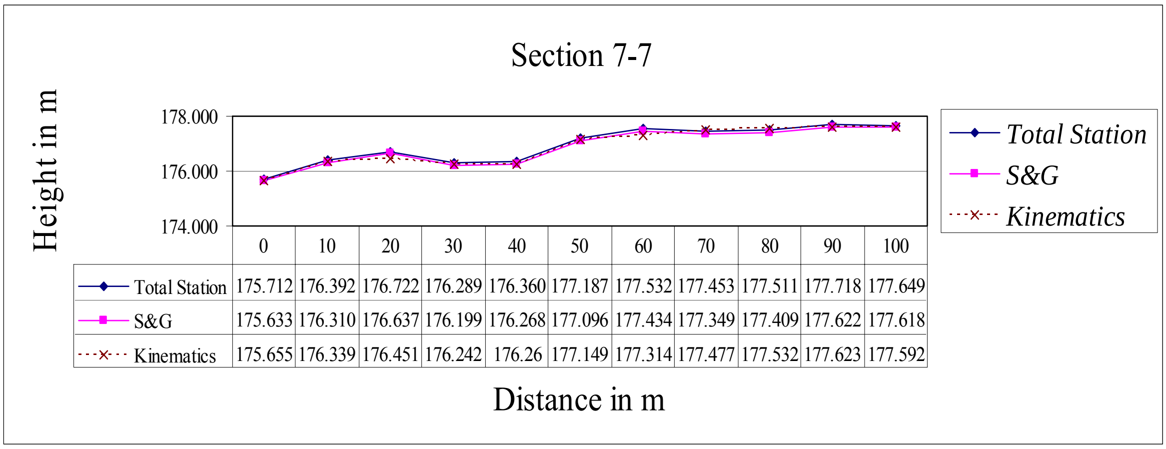


Fig (4-27): Cross Section 7-7

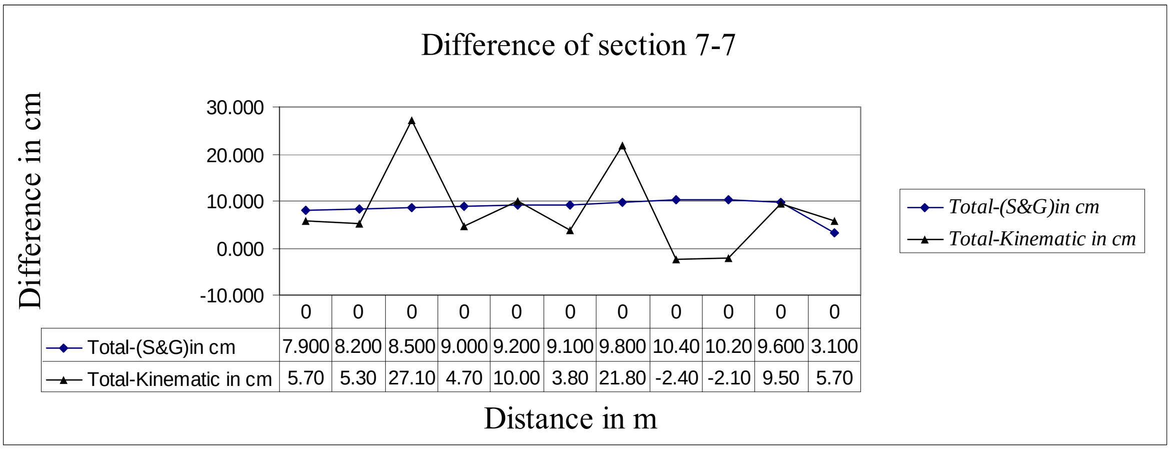


Fig (4-28): Difference of Section 7-7

Table (4-23): Statistics of Cross Section 7-7

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	8.8	8.6	10.4	3.1
Diff.2 in cm	11.8	8.1	27.1	-2.4

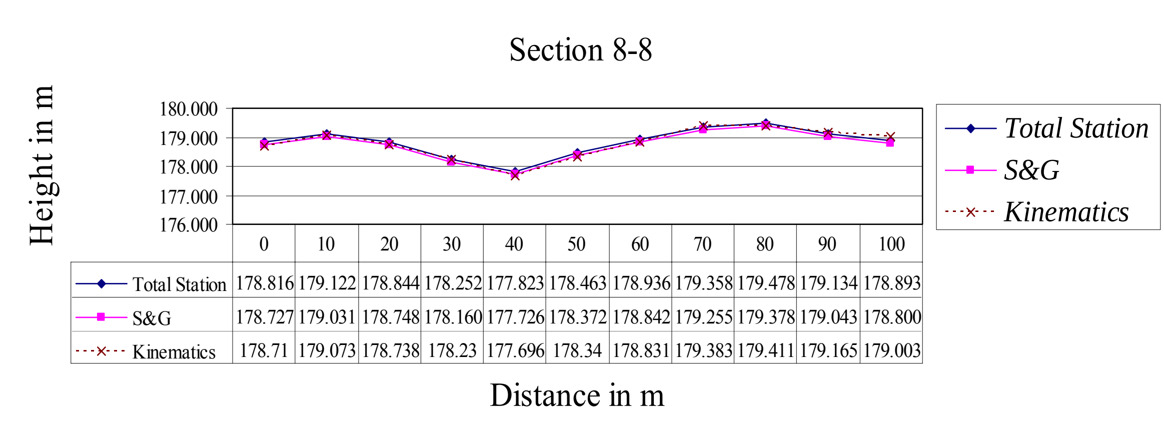


Fig (4-29): Cross Section 8-8

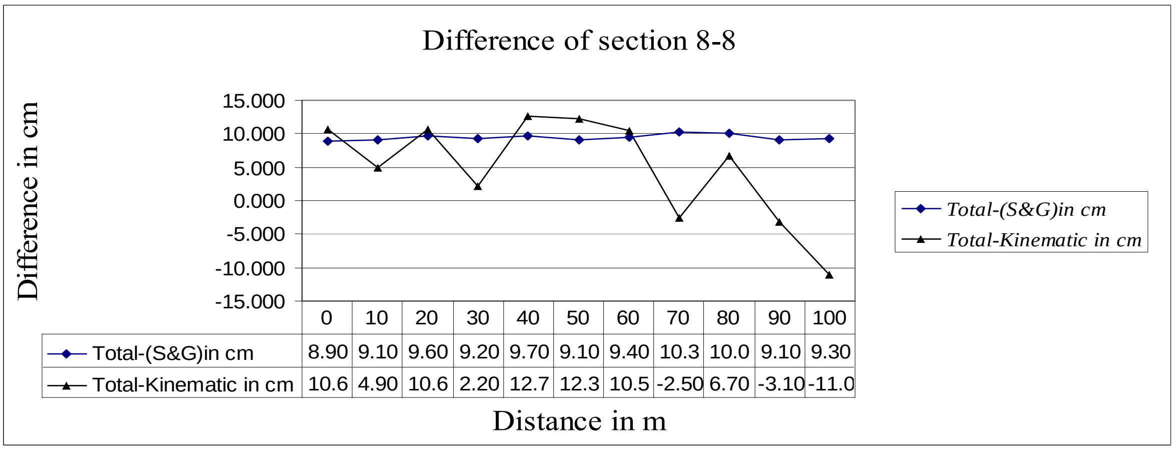
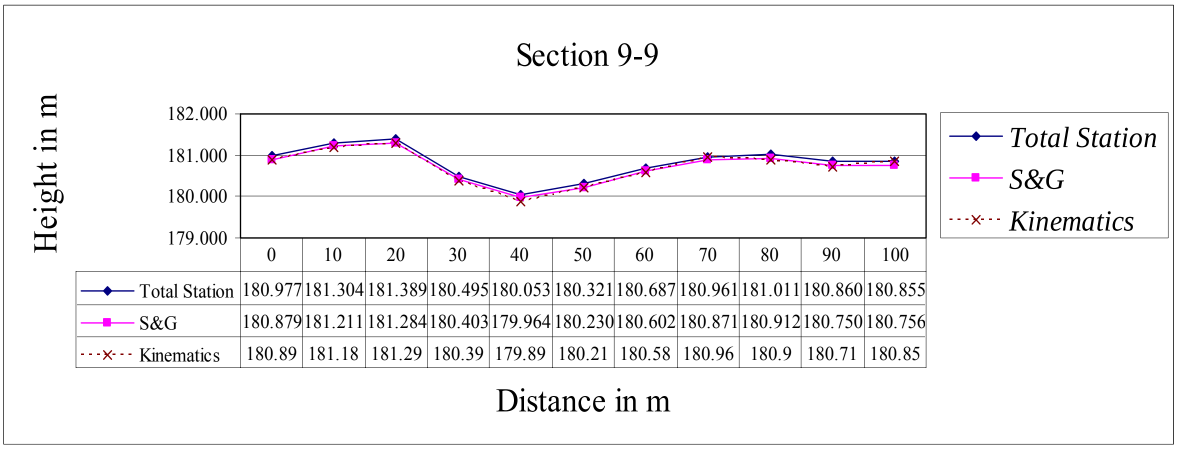


Fig (4-30): Difference of Section 8-8

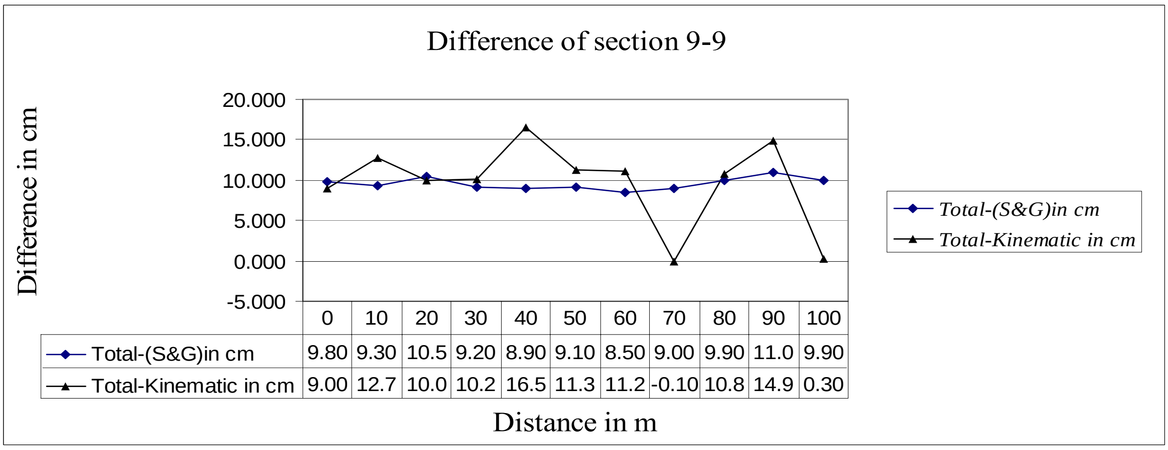
Table (4-24): Statistics of Cross Section 8-8

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.4	9.4	10.3	8.9
Diff.2 in cm	8.8	4.9	12.7	-11.0



	0	10	20	30	40	50	60	70	80	90	100
Total Station	180.977	181.304	181.389	180.495	180.053	180.321	180.687	180.961	181.011	180.860	180.855
S&G	180.879	181.211	181.284	180.403	179.964	180.230	180.602	180.871	180.912	180.750	180.756
Kinematics	180.89	181.18	181.29	180.39	179.89	180.21	180.58	180.96	180.9	180.71	180.85

Fig (4-31): Cross Section 9-9



	0	10	20	30	40	50	60	70	80	90	100
Total-(S&G)in cm	9.80	9.30	10.5	9.20	8.90	9.10	8.50	9.00	9.90	11.0	9.90
Total-Kinematic in cm	9.00	12.7	10.0	10.2	16.5	11.3	11.2	-0.10	10.8	14.9	0.30

Fig (4-32): Difference of Section 9-9

Table (4-25): Statistics of Cross Section 9-9

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.5	9.5	11.0	8.5
Diff.2 in cm	10.9	9.7	16.5	-0.1

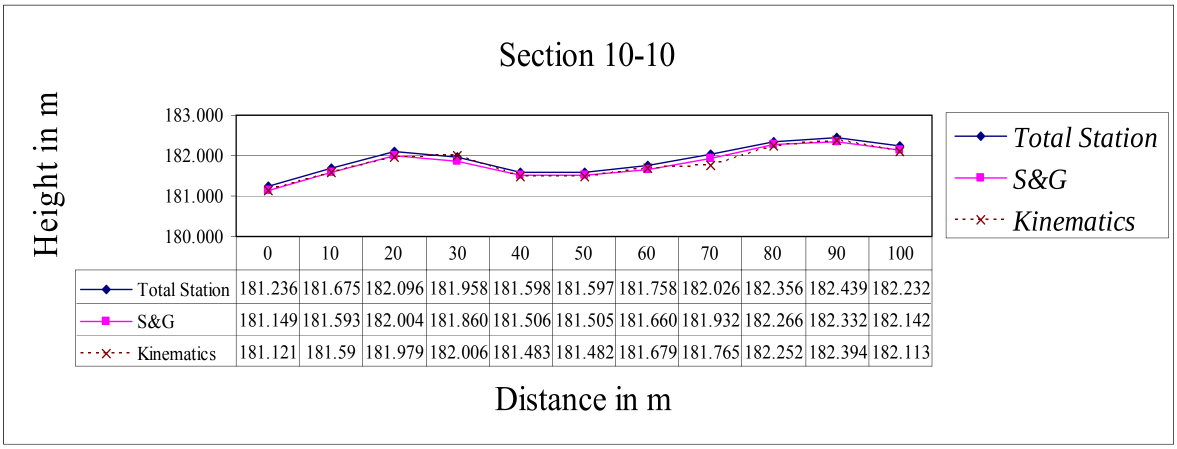


Fig (4-33): Cross Section 10-10

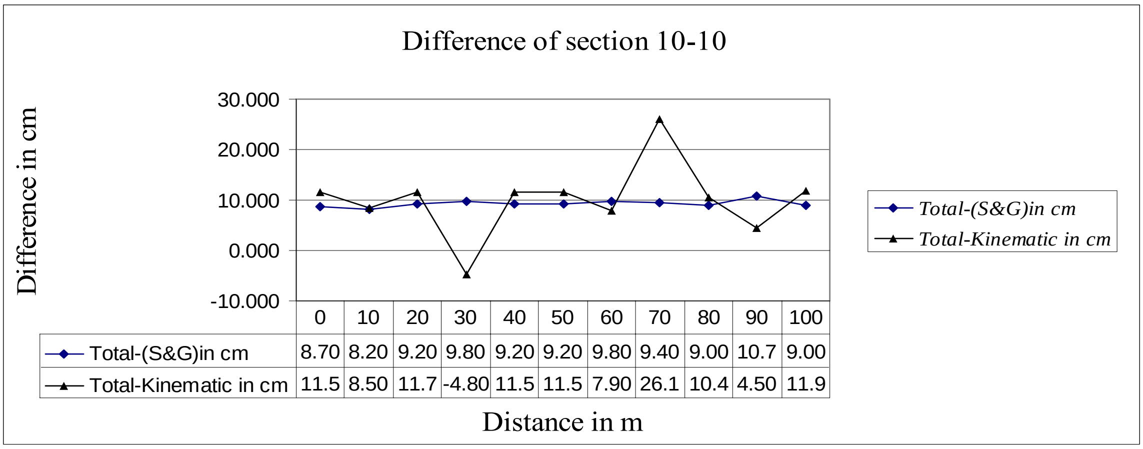


Fig (4-34): Difference of Section 10-10

Table (4-26): Statistics of Cross Section 10-10

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	9.3	9.3	10.7	8.2
Diff.2 in cm	12.2	10.0	26.1	-4.8

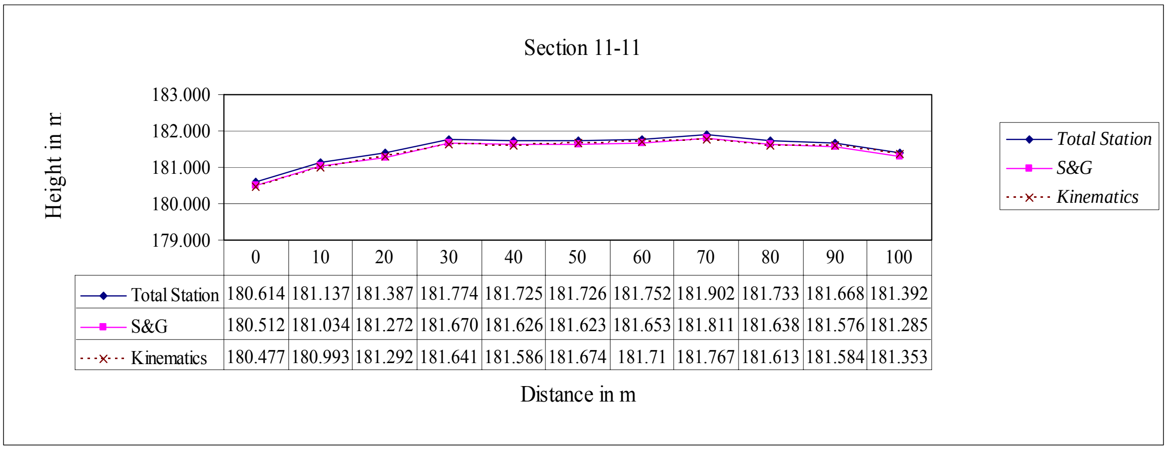


Fig (4-35): Cross Section 11-11

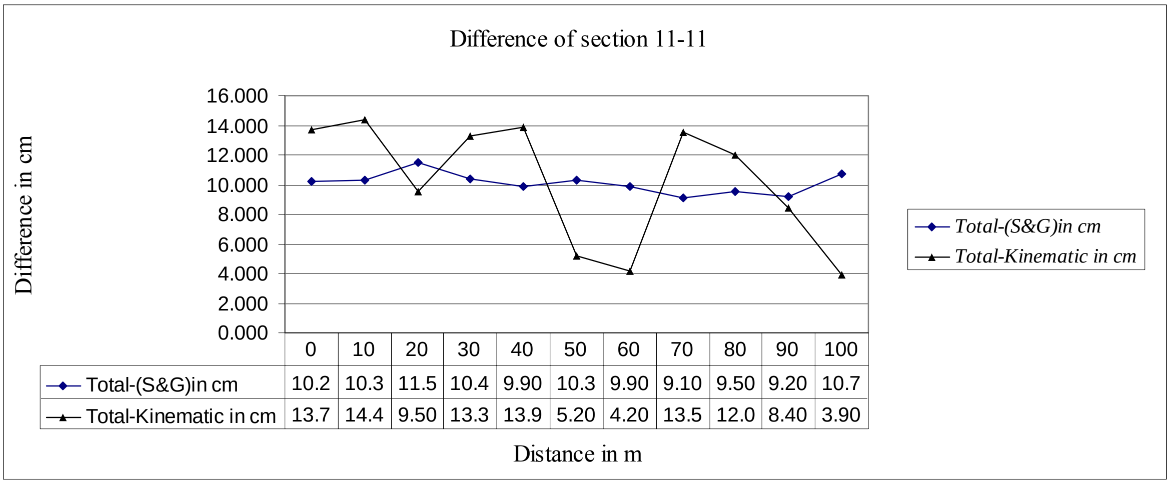


Fig (4-36): Difference of Section 11-11

Table (4-27): Statistics of Cross Section 11-11

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	10.1	10.1	11.5	9.1
Diff.2 in cm	10.9	10.2	14.4	3.9

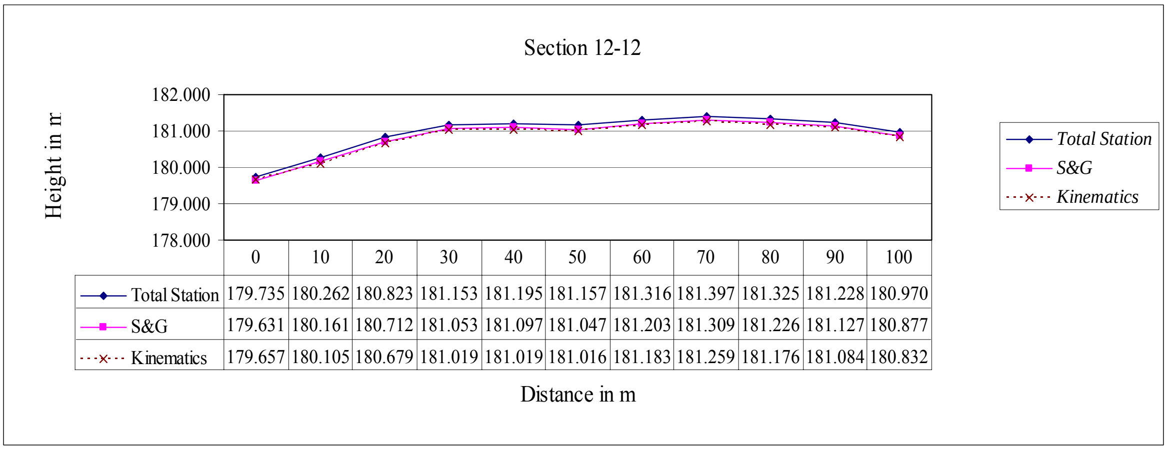


Fig (4-37): Cross Section 12-12

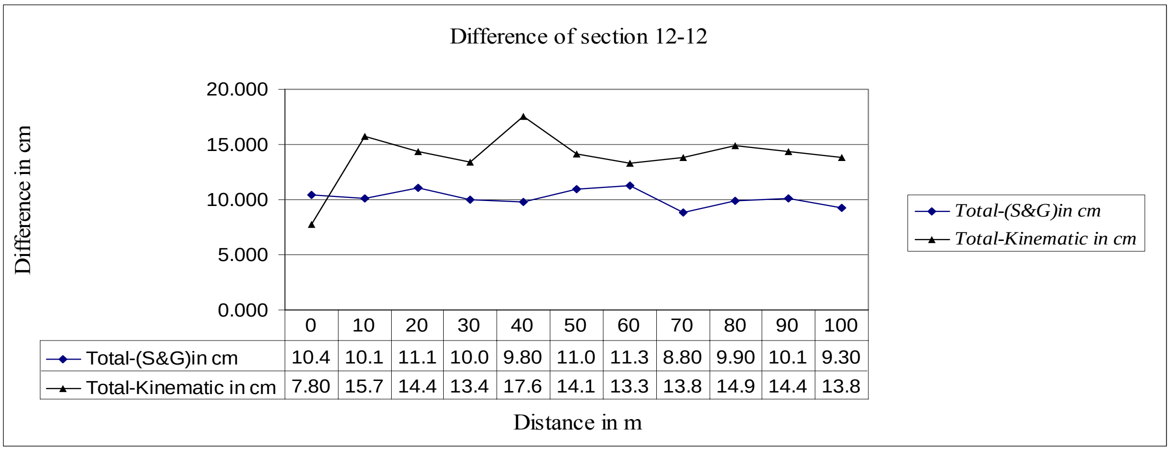


Fig (4-38): Difference of Section 12-12

Table (4-28): Statistics of Cross Section 12-12

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	10.2	10.1	11.3	8.8
Diff.2 in cm	14.1	13.9	17.6	7.8

From table (4-28);

- RMSE values for Stop and Go are in between (8.8 cm to 10.2 cm) but for kinematic are in between (8.8 cm to 14.1 cm).

4.2.2.2 Kinematic GPS Using Moving Car

This part aims to detect the absolute accuracy of survey a segment of the new city EL-SADAKA road- Aswan Governorate. The total length of the road is 1800 m divided in processing to 37 stations, one station / 50 m. This survey done using two techniques of survey as follow:

- Kinematic GPS Survey

This survey was done by using Kinematic GPS by moving car with UTM Zone 36 N and EGM -96 geoid model. The moving car had a speed range of 15 km to 20 km to setup the moving antenna in the road center and to decrease the signal ambiguity and without any initialization start time. The time interval between epochs is 5 seconds in the base receivers and the rover one to get good processing synchronization between the two receivers. [Promark 3.0 manual (2005)] the satellite numbers are from 10 to 12 satellites and the PDOP value is from 1.50 to 2.40.

- Total station Survey

This survey was done by using the base point around the location to have the same GPS coordinates, the surveyed points was in the center of road. The processing data of surveying as shown table (4-29).

- Analysis of Data

The surveying data for GPS and Total Station are in the next table (4-29), to compute the difference between the Total Station and GPS Kinematic, tack Total Station height as a reference data.

Difference Height = Total Station Height – GPS Height. (4-5)

The longitudinal sections of and layout of road are shown in figures (4-40) and (4-41).

Table (4-29): Processing data of Kinematic Surveying

Station No.	Cumulated Distance	Total Station Height	GPS Height	Difference(cm)= Total H-GPS H
1	0	169.838	169.897	-5.8
2	50	170.349	170.362	-1.3
3	100	170.754	170.640	11.3
4	150	171.051	171.020	3.0
5	200	171.526	171.449	7.7
6	250	172.247	172.233	1.3
7	300	172.779	172.659	12.0
8	350	173.680	173.551	12.9
9	400	174.375	174.362	1.3
10	450	175.233	175.238	-0.5
11	500	176.040	176.063	-2.2
12	550	176.763	176.636	12.7
13	600	177.354	177.342	1.2
14	650	177.924	177.847	7.6
15	700	178.544	178.502	4.1
16	750	179.032	179.013	1.8
17	800	179.367	179.328	3.8
18	850	179.821	179.795	2.6
19	900	180.662	180.611	5.0
20	950	181.435	181.370	6.4
21	1000	182.199	182.149	4.9
22	1050	182.563	182.560	0.3
23	1100	182.787	182.801	-1.3
24	1150	183.271	183.281	-1.0
25	1200	183.609	183.646	-3.7
26	1250	183.897	183.907	-1.0
27	1300	184.048	184.012	3.5
28	1350	184.334	184.362	-2.7
29	1400	184.398	184.417	-1.9
30	1450	184.467	184.471	-0.3
31	1500	184.507	184.521	-1.3
32	1550	184.600	184.653	-5.3
33	1600	185.257	185.279	-2.1
34	1650	186.157	186.230	-7.3
35	1700	186.396	186.460	-6.3
36	1750	186.244	186.382	-13.7
37	1800	186.043	186.118	-7.4

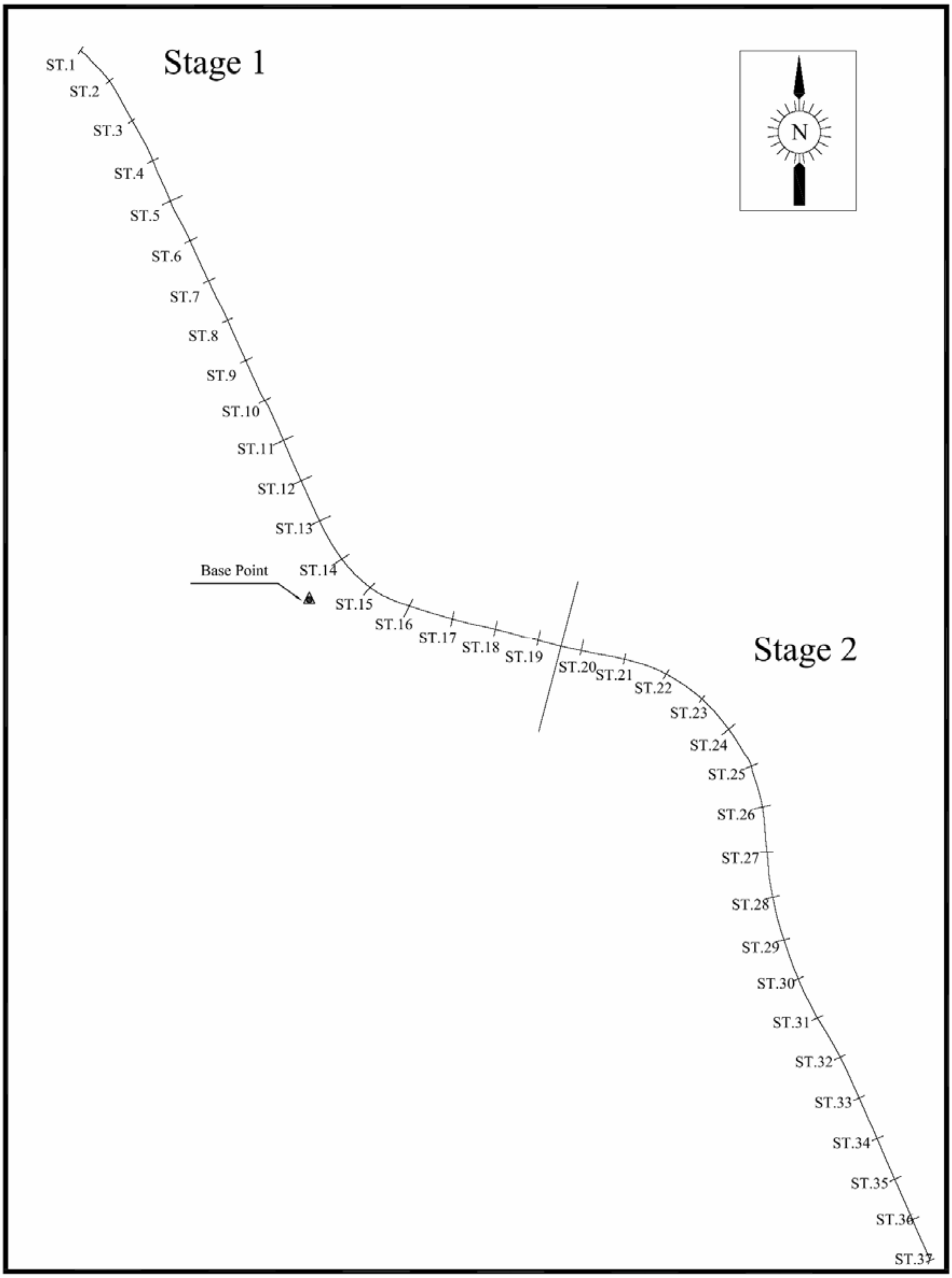
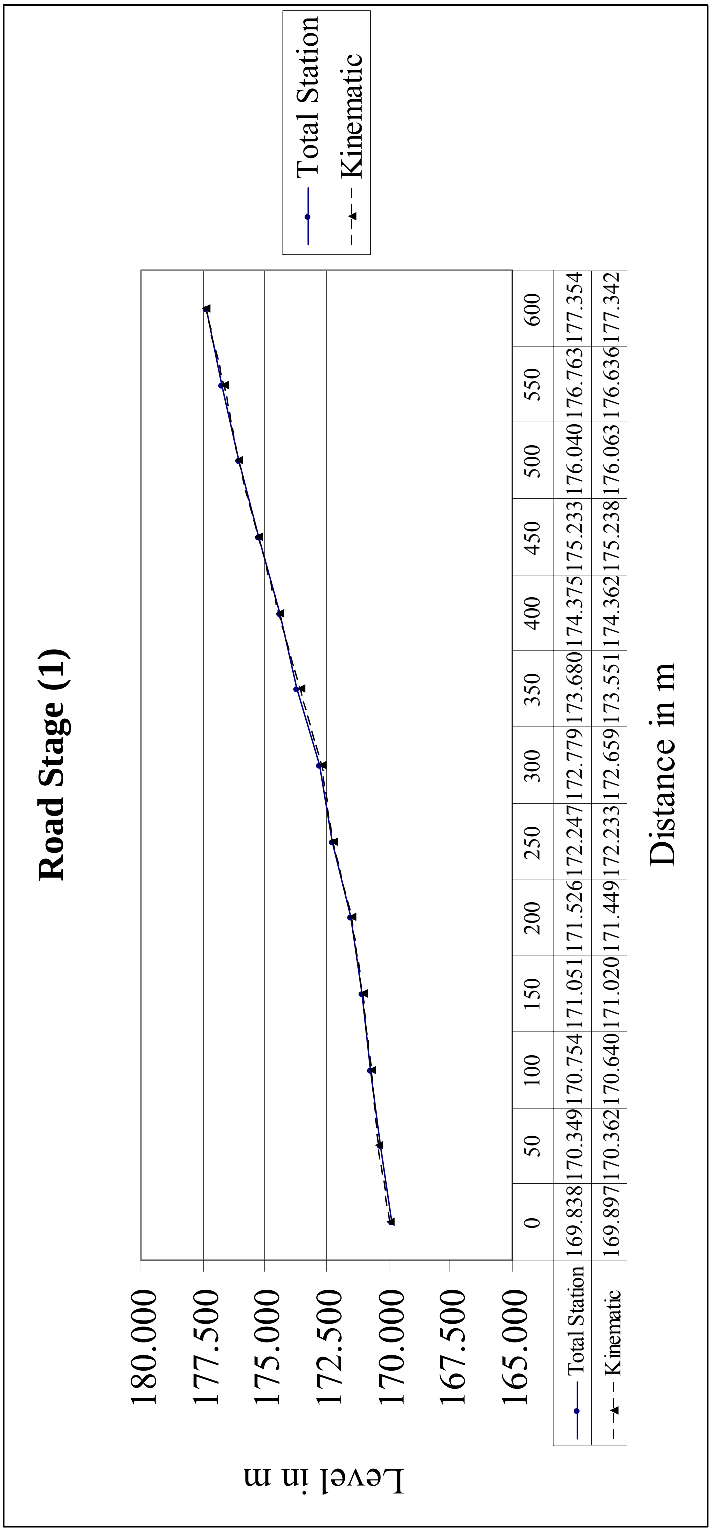
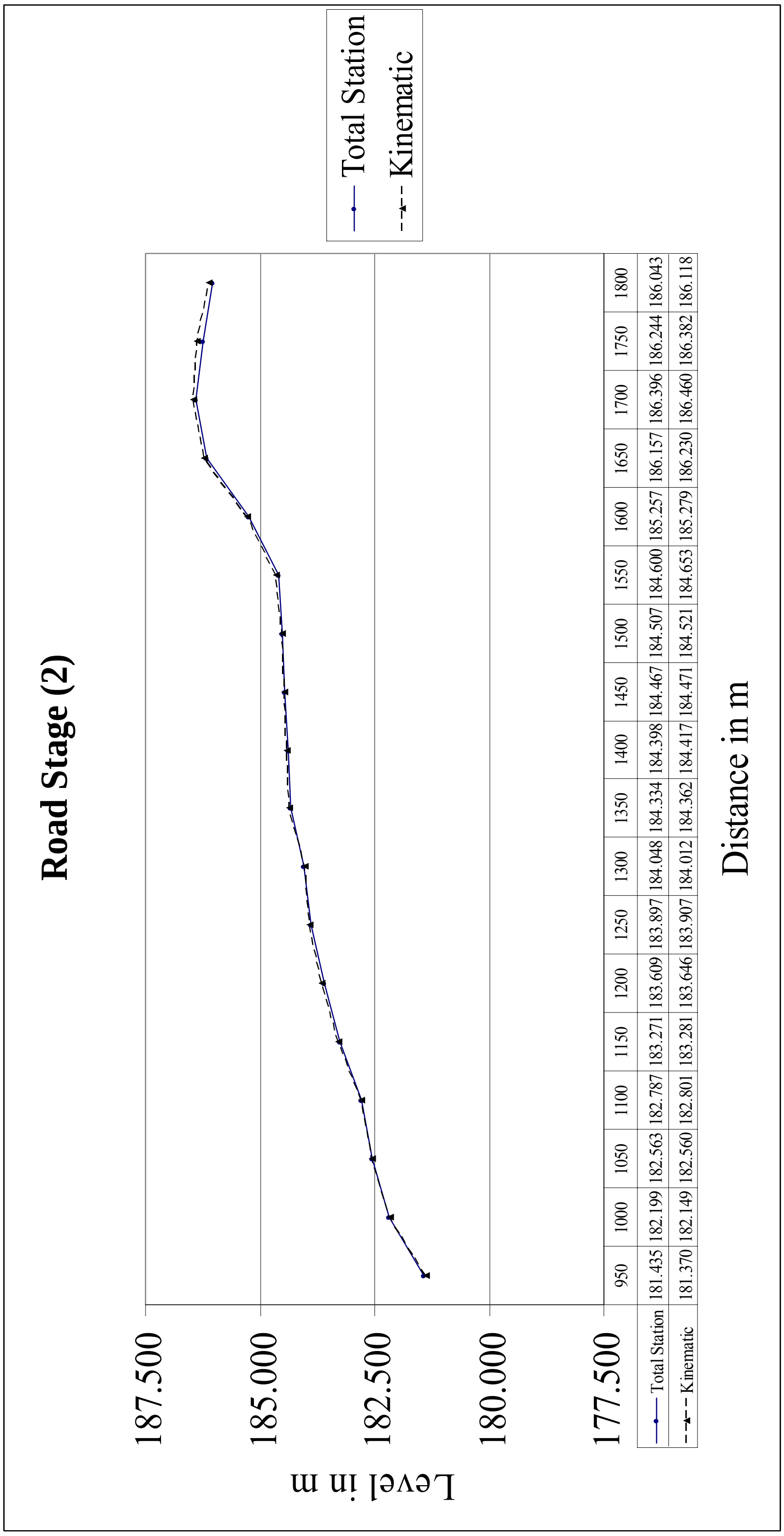


Fig. (4-39)Layout of Road



	0	50	100	150	200	250	300	350	400	450	500	550	600
Total Station	169.838	170.349	170.754	171.051	171.526	172.247	172.779	173.680	174.375	175.233	176.040	176.763	177.354
Kinematic	169.897	170.362	170.640	171.020	171.449	172.233	172.659	173.551	174.362	175.238	176.063	176.636	177.342

Fig (4-40): Road Stage (1)



	950	1000	1050	1100	1150	1200	1250	1300	1350	1400	1450	1500	1550	1600	1650	1700	1750	1800
Total Station	181.435	182.199	182.563	182.787	183.271	183.609	183.897	184.048	184.334	184.398	184.467	184.507	184.600	185.257	186.157	186.396	186.244	186.043
Kinematic	181.370	182.149	182.560	182.801	183.281	183.646	183.907	184.012	184.362	184.417	184.471	184.521	184.653	185.279	186.230	186.460	186.382	186.118

Fig (4-41): Road Stage (2)

From table (4-29); the statistics of data analysis as shown in the following table (4-30)

the statistics of data analysis *Table (4-30): Statistics Road Leveling Data*

	RMSE cm	Mean value cm	Max. value cm	Min. value cm
Diff.1 in cm	6.0	1.0	12.9	-13.7

From table (4-30);

- The RMSE of the difference between the Total Station Level and Kinematic Level is equal to 6.00 cm with a mean value of 1.0 cm. This proves the quality of using GPS technology for such application.
- The difference values are in between 12.9 cm as Maximum value and -13.7 cm as Minimum value.

4.3 Analysis and Discussion

Using GPS technology for surveying applications safes cost and time, this is concluded using this study as well as previous studies.

GPS offer different techniques that will be suitable for different surveying applications such as:

Static GPS technique: is suitable for;

- Establishing control points as well as observing triangulation networks.

- Monitor the earth crustal movements in areas where there are high-importance civil engineering structures such as Aswan-High Dam.
- Cadastral surveying

Kinematic GPS techniques (stop&go and kinematic): is suitable for;

- DEM Generation
- Generation of longitudinal and cross sections for roads and irrigations canal& drains.

GPS technology proved its high quality behaviour for different surveying applications during this research comparing with classical surveying techniques. The accuracy offered by GPS technology will be much better in the near future with the launch of new civilian control system (Galileo) as well as a full constellation capabilities offered by Glonass. With three different constellations will be in use with more signals and a number of visible satellites not less than 75, the improvement in positioning accuracy is promising and inevitable.

4.3.1 Static GPS Technique

a. Group (1):

Under the conditions of visible satellite number was not less than 6 satellites and PDOP value was not exceed 2.70.

- For baselines up to 10 km, the best elapsed time for observation is nearly equal to (10 min+1 min. /km)
- For baselines 10 km to 20 km, the best elapsed time for observation is nearly equal to (15 min+1.5 min. /km).

- For baselines from 5 km to 10 km; increasing elapsed time of 10 min. increase the relative values of δ' East, δ' North, δ' H and δ' L.
- Increasing the elapsed time of the previous reference time not necessary increase the accuracy of relative error (95% Confidence).
- The value of the relative error of the base line (95% Confidence) equal to 5 ppm (5 mm/km) as example for base line 5 km, the relative error equal to 25 mm.
- For baseline 0 to 5 km; there isn't any change of using the two types of ephemeris (broadcast & IGS final)
- For baseline 10 to 20 km; δ' East value is smaller than δ' North value and smaller than δ' H values. This difference reaches to 11mm in East, 34 mm in North and 38 mm in H as a maximum values.
- For all baselines; the accuracy of maximum time for final precise ephemeris is the same of broadcast one.
- The error in coordinates of stand alone in East is in between (+0.104 to -2.807 m), for North equal to (+1.511 to -1.815 m) and for H is in between (+2.096 to -4.54 m) for all observation time.
- The error values in stand alone technique exceed for H coordinates than others coordinates.
- For baseline up to 10 km; increasing the elapsed time doesn't increase the accuracy where there is no sequence of difference change according to increasing time.
- For baseline from 10 to 20 km; increasing the elapsed time increases the accuracy of coordinates.

b. Group (2):

- The value of the relative error of the base line (95% Confidence) equal to 5 ppm (5 mm/km) which is 15 cm.
- The suitable observation time with acceptable accuracy is 60 min under conditions of 8 visible satellites and PDOP of not more than 2.
- The suitable observation time with acceptable accuracy is 90 min or more under conditions of less than 6 visible satellites and PDOP of more than 2.
- In increasing the elapsed time; there is an improvement in δ'East, δ'North and δ'L values.
- Using Broadcast ephemeris for processing baselines gives acceptable accuracy under good conditions of visible satellites and PDOP values.
- For less number of visible satellites than 6 and PDOP larger than 2, the use of IGS-precise ephemeris is recommended.

4.3.2 Kinematics GPS Technique.

a. DEM generation by Total Station, (Stop and Go) and Kinematic GPS techniques.

- The height relative error of Stop and Go technique (95% Confidence) is in between 0.30 cm to 0.60 cm but for kinematic technique is in between 3.4 cm to 10.2 cm.
- The difference of S&G and Kinematic technique about Total Station technique is in cm level but for Stand alone in meter level, that for the differential solution of the S&G and Kinematic techniques, so stand alone isn't have any suitable solution for height.
- The Stop and Go technique height had a difference from Total Station height values in between (3.1 cm to 11.6 cm) and RMSE of (9.7cm)

for all data, but for the cross sections RMSEs in East –West are in between (9.5 cm to 10.0 cm) and for North – South are in between (8.8 cm to 10.2 cm).

- The Kinematic technique height had a difference from Total Station height values in between (-14.5 to 25.5 cm) and RMSE of (12.05 cm) for all data, but for the cross sections RMSEs in East – West are in between (11.6 cm to 13.9 cm) and for North – South are in between (8.8 cm to 14.1 cm).
- From percentage of the cut and fill volumes; the arrangement of observation techniques as follow Total Station (10 m), Total Station (20 m), Kinematic (10 m), Stop and Go (10m), Stop and Go (20m) and kinematic (20 m).

b. Kinematic technique by using moving car.

In the conditions satellite numbers not less than 10 and PDOP value not exceed 2.40.

- The difference values are in between 12.9 cm as Maximum value and -13.7 cm as Minimum value, the average difference can be equal to 1.0 cm.
- The RMSE of the difference between the Total Station Level and Kinematic Level is equal to 6.0 cm.
- Using EGM-96 as a geoid model in South Egypt (Aswan as a case study) can get suitable accuracy of ortho-metric height in comparing with the total Station height.
- The suitable speed of GPS moving car to survey is in between 15- 20 km per hour.

Chapter 5

CONCULUSIONS

and

RECOMMENDATIONS

5.1 Conclusions

The main objective of this thesis is to reflect south Egypt characteristics that affects GPS technology behavior, where it presents accuracy assessment for GPS engineering applications such as Positioning accuracy and DEM generation by using GPS techniques (Static, (Stop and Go) and kinematic) in Aswan city .

This research included two parts of filed work using single frequency receivers which are;

- Static GPS Technique
    a. Group (1); for baselines length up to 20 km.
        - Change the baseline length.
        - Change elapsed time for every baseline.
        - Change processing ephemeris.
        - Compare the differential solution with the Stand alone solution.
    b. Group (2); for baseline length of 31 km.
        - Change elapsed time.
        - Change processing ephemeris.
        - Compare the differential solution with the Stand alone solution.

- Kinematics GPS Technique.
    a. DEM generation by Total Station, Stop and Go and Kinematic.

b. Road longitudinal section generation using GPS Kinematic technique by set up the receiver on a moving car.

From analysis of the obtained results, the followings can be concluded.

5.1.1 Static GPS Technique

a. Group (1):

Under the conditions of visible satellite number was not less than 6 satellites and PDOP value was not exceed 2.70.

- The recommended elapsed observation time for baselines up to 10 km is equal to (10 min+1 min. /km)
- The recommended elapsed observation time for baselines (10 km to 20 km) is equal to (15 min+1.5 min. /km).
- The value of the relative error of the baseline (95% Confidence) equal to 5 ppm (5 mm/km).
- The use of IGS final ephemeris is recommended where broadcast ephemeris can contain errors (RMS of satellite 3D-position is larger than 6.00 m)
- The results of using of broadcast ephemeris or IGS final ephemeris could be similar, if there is high number of visible satellite (more than 7 or 8 satellites) with good DOP value (less than 2.00) as well as acceptable RMS of satellite 3D-position less than 3.00 m (for broadcast ephemeris).

b. Group (2): (baseline of 31 km.)

- The baseline relative error (95% Confidence) is equal to 5 ppm (5 mm/km)
- The recommended observation time is 60 min under conditions of 8 visible satellites and PDOP of no more than 2.
- The results of using of broadcast ephemeris could be acceptable if there is high number of visible satellite (more than 7 or 8 satellites) with good DOP value (less than 2.00) as well as acceptable RMS of satellite 3D-position less than 3.00 m.
- The use of IGS final ephemeris is recommended where broadcast ephemeris can contain errors (RMS of satellite 3D-position is larger than 6.00 m)

5.1.2 Kinematics GPS Technique.

a. DEM generation using Total Station, (Stop and Go) and Kinematic GPS techniques.

- The height relative error of (Stop and Go) technique (95% Confidence) is in range of (0.30 cm to 0.60 cm) but for kinematic technique is in between (3.40 cm to 10.20 cm).
- The difference of (Stop&Go) and Kinematic techniques comparing with classical surveying (total station) is in cm level but for Stand alone is in meter level. So the use of standalone GPS technique is only recommended for preliminary investigation purposes.
- The Stop and Go technique height had a difference from Total Station height values in range (3.1 cm to 11.6 cm) and RMSE of (9.7 cm) for all data,

- The RMSEs for cross sections in (East –West) direction are in range of 9.5 cm to 10.0 cm and for (North – South) direction cross sections are in range of 8.8 cm to 10.2 cm using (stop& go) technique.
- The Kinematic technique height had a difference from Total Station height values in range (-14.5 to 25.5 cm) and RMSE of (12.0 cm) for all data.
- The RMSEs for cross sections in (East –West) direction are in range (11.6 cm to 13.9 cm) and for (North – South) direction cross sections are in range (8.8 cm to 14.1 cm) using kinematic technique.
- There is strong agreement between (stop&go) and kinematic GPS techniques in the percentage of the cut and fill volumes.

b. Kinematic technique by using moving car.

In the conditions satellite numbers not less than 8 and PDOP value not exceed 2.40. The reference height data using classical surveying (total station)

- The differences in height are in range (12.9 to -13.7) cm, with an average value of 1.0 cm.
- The RMSE of the difference between the Total Station Level and Kinematic Level is equal to 6.0 cm.

5.2 Future Works

Based on the results of the previous work, the following points could be investigated in the future:

- Improving the GPS ortho-metric heights by using another local Geoid Model or using the last version of Global Geoid Model (EGM 2008).
- Improving the observed GPS coordinates in Aswan region by calculate the parameters of translate WGS-84 Coordinate System to Egypt Coordinate System.
- Study the accuracy of using GLONASS Russian Satellite System in the surveying applications in south Egypt.

5.3 Publications

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Appendix A

Specifications of used Instruments & Softwares

The following surveying instruments and software were used:

- **ProMark 3.0 Single Frequency GPS system Specifications:-**

This receiver is Copy Right Thales Navigation Company from 2005 and this receiver has the following specifications see table (A-1)

Table (A-1): Specifications of Receivers.

[Promark 3.0 manual (2005)]

Parameter	Specification
GPS survey mode supported	Static, Stop-and-go, Kinematic
Survey accuracy (RMS) - Static	Horizontal: 0.005m + 1 ppm Vertical: 0.010m + 2 ppm
Survey accuracy (RMS) – Stop-and-go	Horizontal: 0.012m + 2.5 ppm Vertical: 0.015m + 2.5 ppm
Real-Time Performance	SBAS (WAAS/EGNOS) RMS: Horizontal < 1 meter (3 feet) DGPS (Beacon or RTCM) RMS: Horizontal < 1 meter (3 feet)
Survey point spacing – Static (vector length)	Up to 20 kilometers
Survey point spacing – Stop and- go (vector length)	Up to 10 kilometers

Parameter	Specification
Observation time - Static	4 to 40 minutes typical, depending upon vector length
Observation time – Stop-and go	15 seconds typical
Initialization time – Stop-and go	15 seconds on known points 5 minutes on initializer bar
GPS satellite channels	12
SBAS satellite channels	2
GPS satellite elevation mask	10 degrees
Recording interval	1 – 30 seconds
Operating temperature range	-10 to +60 degrees C
Battery type	3.7 V Li-Ion, 3900 mAh
Battery life	8 hours (typical operation)
Data storage memory capacity	128 MB SDRAM, 128 NAND Flash Memory Removable SD Card: up to 1 GB

Fig. (A-1): Promark 3.0 Receiver.

[Promark 3.0 manual (2005)]

Fig. (A-2): Components of Receiver Bag.

[Promark 3.0 manual (2005)]

Fig. (A-3): Promark 3.0 in Static Technique.

[Promark 3.0 manual (2005)]

Fig. (A-4): Initialization by Bar for Kinematic Technique.

[Promark 3.0 manual (2005)]

- **Total Station TOPCON GTS-712 Specifications:-**

Table (A-2): Specifications of Total Station.

[TOPCON Manual (2000)]

Parameter		Specification
TELESCOPE	Objective Lens	45mm (1.77in.), EDM: 50mm (1.96in.)
	Magnification (X)	30X
	Resolving Power	2.5”
	Minimum Focus	1.3m
	Illumination	Provided
DISTANCE MEASUREMENT	Measuring Range (normal conditions)	Mini prism= 900 m (3000 ft) 1 prism= 2200m (7200ft) 3 prisms= 2900m (9500ft) 9 prisms= 3600m (11800ft)
	Accuracy	±(2mm+2ppm) m.s.e
	Measuring Time	Single-Repeat = 2.5 seconds Tracking = 0.6 seconds
	Minimum Reading	Fine: 1mm (0.005ft.)/0.2mm (0.001ft.) Coarse: 1mm (0.005ft.) Tracking: 10mm (0.02ft.)
	Atmospheric Correction	-999.9ppm to +999.9 (0.1ppm steps)
	Prism Constant Correction	-99.9mm to +99.9 (0.1mm steps)
	Ambient Temperature Range	-20° C to +50° C -4°F to +122°F

Parameter		Specification
ANGLE MEASUREMENT	Minimum Reading	1” or 5” (1.0mgon)
	Accuracy	3”
	Detection Method	H: 2sides V: 2 sides
	Circle Diameter	71 mm
TILT SENSOR	Type	Dual-axis Liquid Type
	Range	±3’
LEVEL SENSITIVITY	Circular Level	10’/2mm
	Plate Level	30”/2mm
OPTICAL PLUMMET TELESCOPE	Magnification	3X
	Focusing Range	0.5mm to infinity
BATTERY PACK	Output Voltage	7.2 volts
	Capacity	2.8AH
	Operating Time	Distance Measurement = 4.5 hours Angle Measurement = 10 hours Recharging Time = 1.5 hours
	Battery	2.2 lbs. (1.0kg)
BATTERY CHARGER	Input Voltage	AC 120V (BC-20B)
	Recharging Time	1.5 hours
	Operating Temperature	50°F to 104°F
	Charging Signal	Red Lamp
	Weight	1.1 lbs. (0.5kg)

Parameter		Specification
DIMENSIONS	Instrument (with battery)	11.6(H) x 8.3(W) x 6.4(D)
	Weight	15.1 lbs. (6.9kg)
	Carrying Case	8.1 lbs. (3.7kg)

Fig. (A-5): TOPCON GTS-712.

[CLEARY (2009)]

- **GNSS Solution Software Version 3.00.07:-**

This program is a copy right of Magellan Navigation Company; GNSS Solutions is the indispensable software tool for all surveyors who need to be efficiently and smoothly assisted in their surveys. GNSS Solutions really offers high standards of performance, processing speed, compactness and flexibility. GNSS Solutions can support a wide range of surveying applications, whether conducted in post-processing or real time. What is more, GNSS Solutions is capable of handling post-processing and real-time data within the same project.

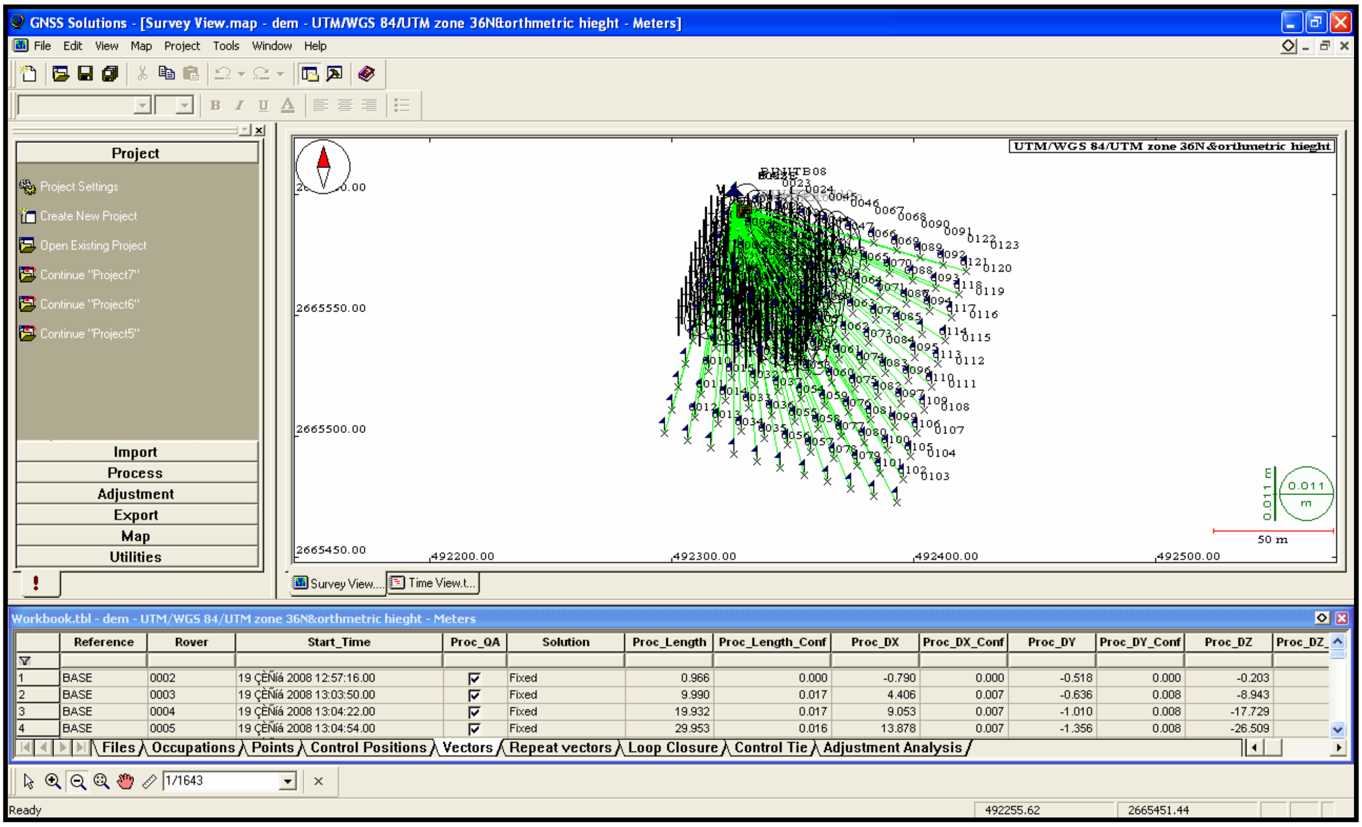


Fig. (A-6): GNSS Solution Survey View.

[GNSS Solution (2007)]

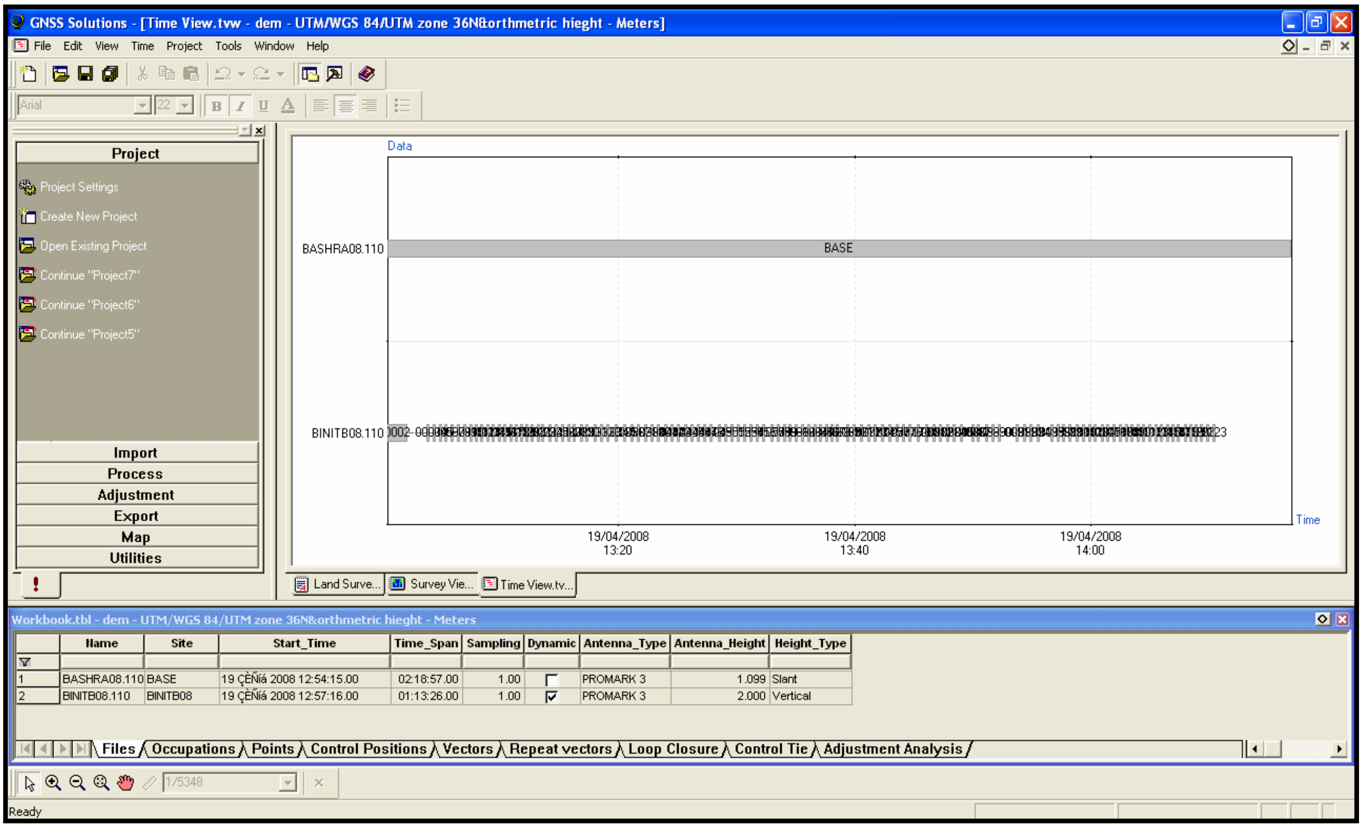
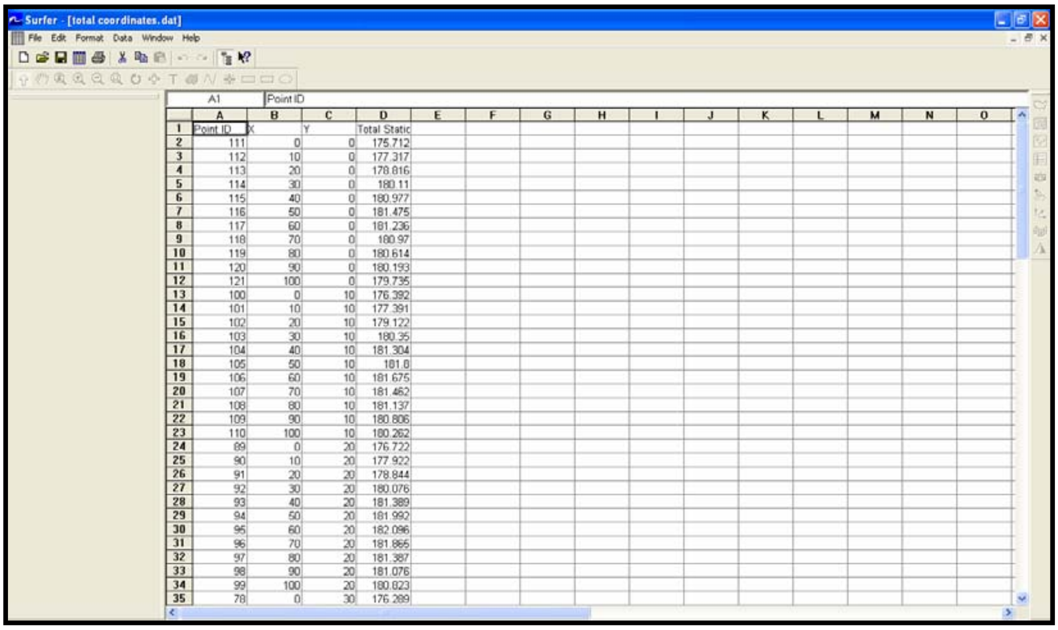


Fig. (A-7): GNSS Solution Time View.

[GNSS Solution (2007)]

- **Golden SURFER Package Version 8.0:-**

Golden Surfer is important software to draw the contour maps, grids, post maps, 3D maps surfaces, base maps and to compute the cut and fill volumes as shown in the following figures. [Golden Software Inc. (2002)]



	A	B	C	D
1	Point ID	X	Y	Total Static
2	111	0	0	175.712
3	112	10	0	177.317
4	113	20	0	178.816
5	114	30	0	180.11
6	115	40	0	180.977
7	116	50	0	181.475
8	117	60	0	181.236
9	118	70	0	180.97
10	119	80	0	180.614
11	120	90	0	180.193
12	121	100	0	179.735
13	100	0	10	176.392
14	101	10	10	177.391
15	102	20	10	179.122
16	103	30	10	180.35
17	104	40	10	181.304
18	105	50	10	181.8
19	106	60	10	181.675
20	107	70	10	181.462
21	108	80	10	181.137
22	109	90	10	180.806
23	110	100	10	180.262
24	89	0	20	176.722
25	90	10	20	177.922
26	91	20	20	178.844
27	92	30	20	180.076
28	93	40	20	181.389
29	94	50	20	181.992
30	95	60	20	182.096
31	96	70	20	181.865
32	97	80	20	181.387
33	98	90	20	181.076
34	99	100	20	180.823
35	78	0	30	176.289

Fig. (A-8): Surfer data sheet.

[Golden Software (2002)]

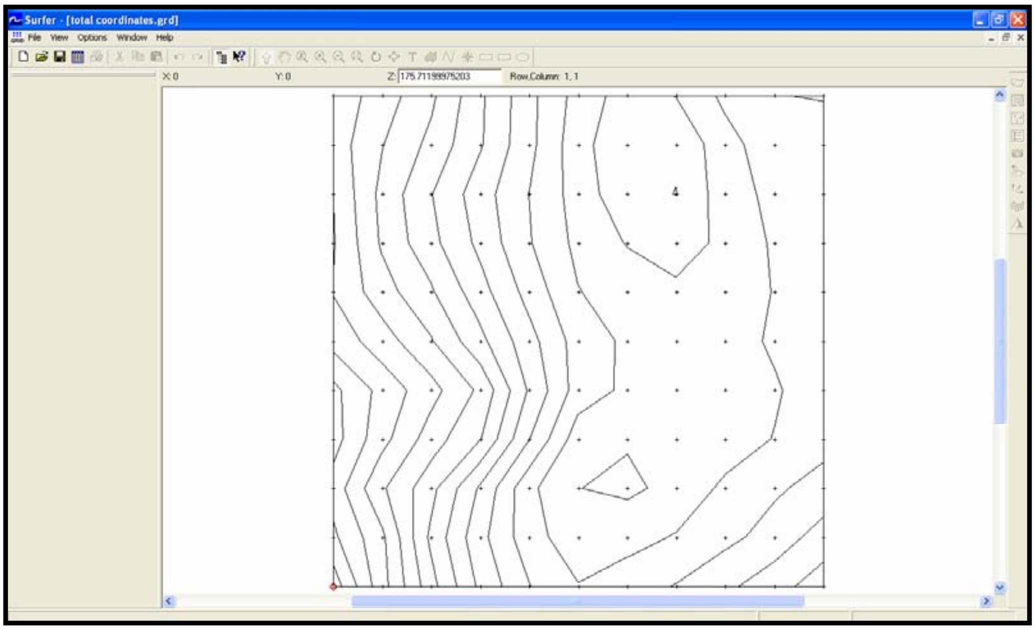


Fig. (A-9): Grid plot of Surfer.

[Golden Software (2002)]

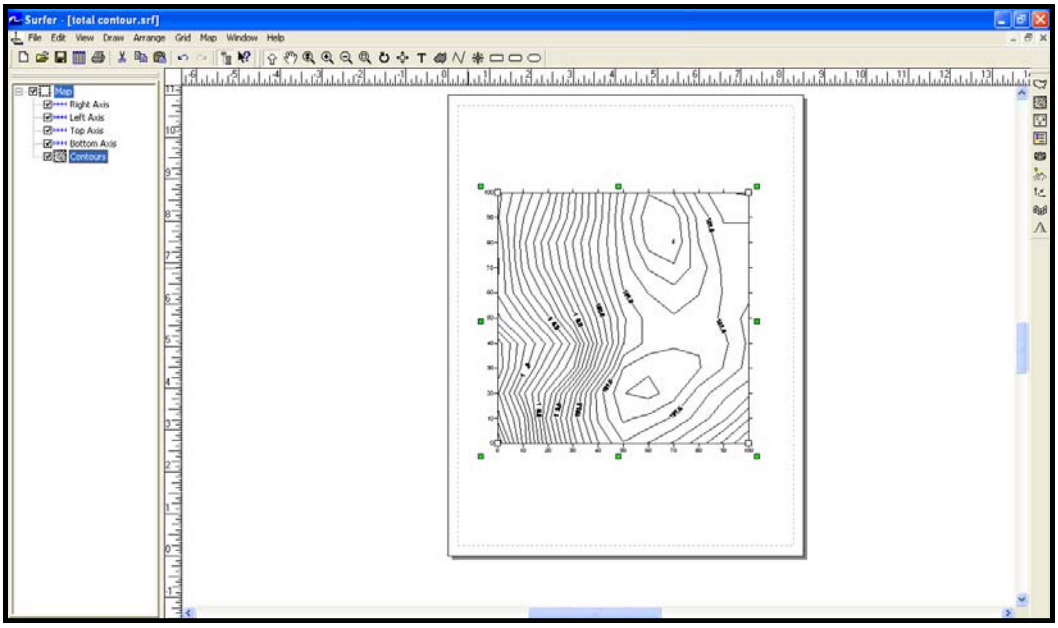


Fig. (A-10): Contour plot of Surfer.

[Golden Software (2002)]

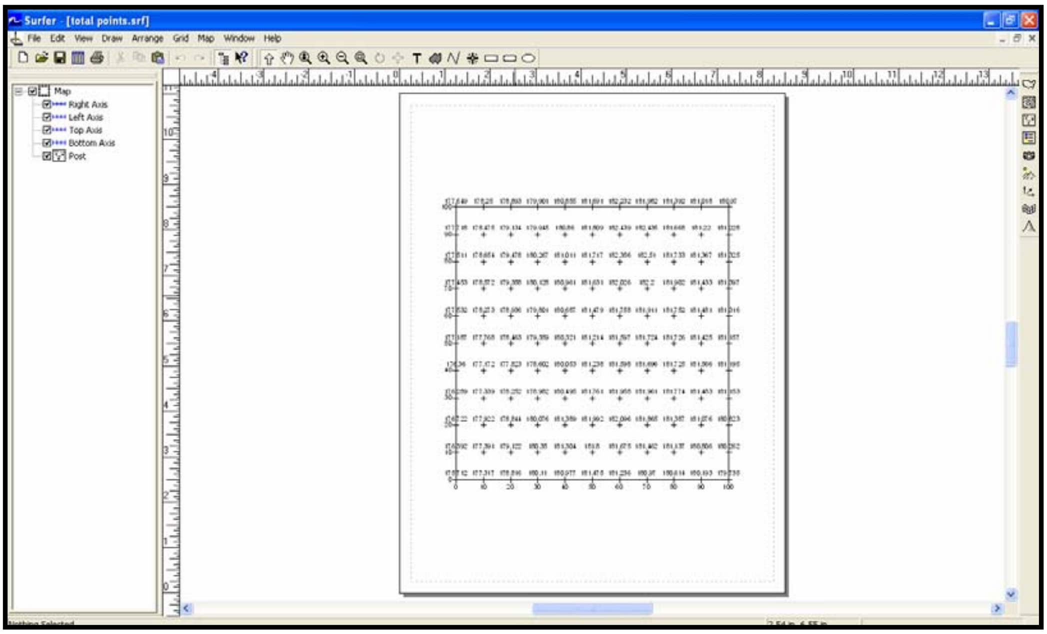


Fig. (A-11): Base map of Surfer.

[Golden Software (2002)]

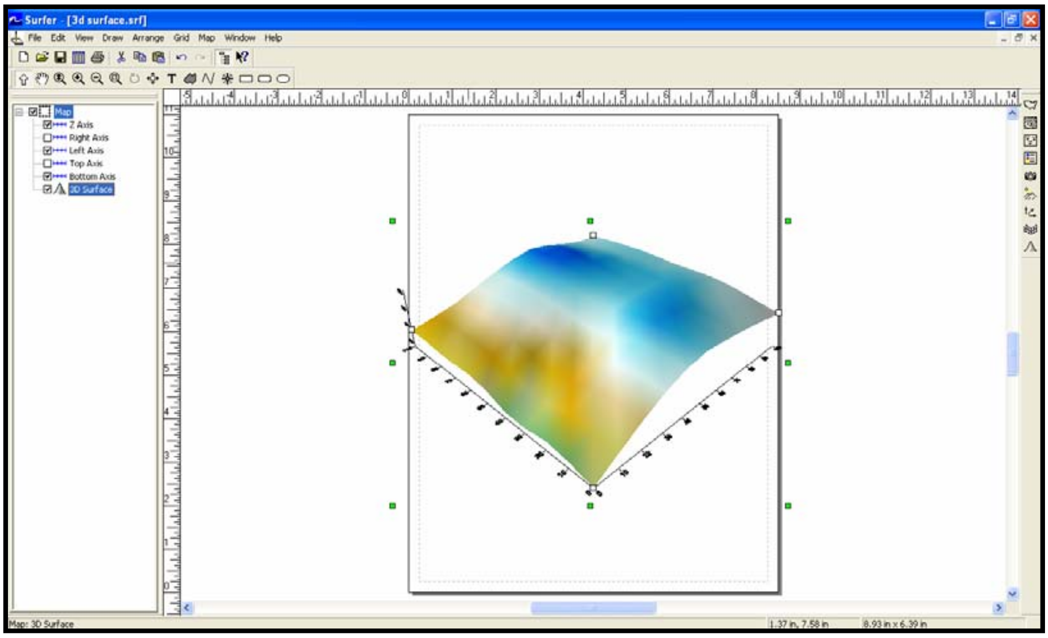


Fig. (A-12): Wire frame map.

[Golden Software (2002)]

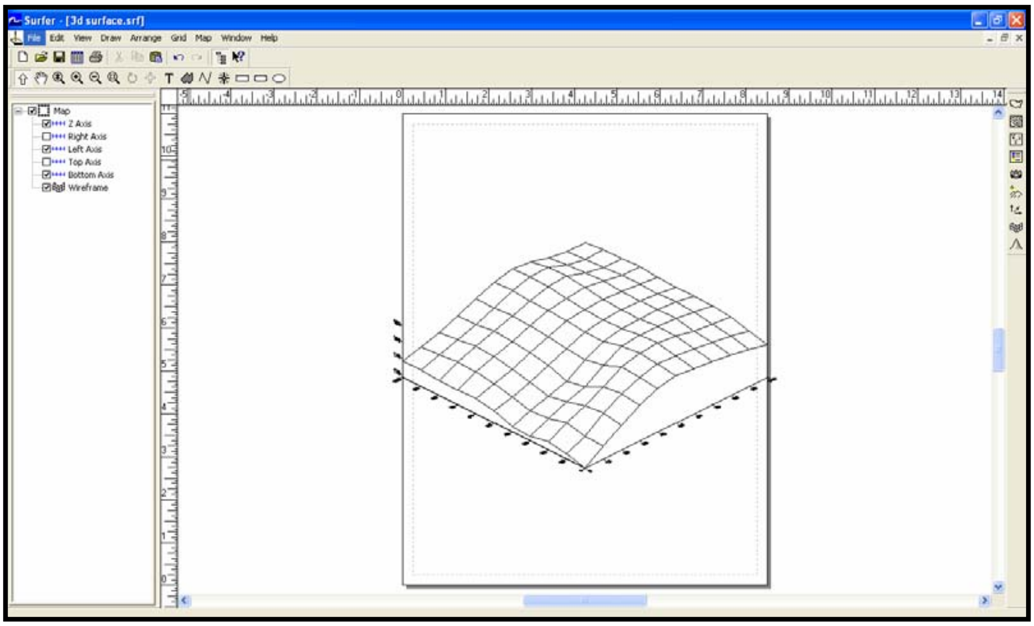


Fig. (A-13): 3D-Surface map.

[Golden Software (2002)]

Appendix B

GPS broadcast ephemeris accuracy statistics.

Table (B-1): GPS broadcast ephemeris accuracy statistics .
(compared with IGS final orbit) [(GPS Lab, 2009)]

Year 2007, Day 109 (19/4/2007)

PRN	X [m]			Y [m]			Z [m]			3D [m]		
	min	max	r.m.s	min	max	r.m.s	min	max	r.m.s	min	max	r.m.s
1	-4.876	2.925	1.666	-1.534	2.444	1.197	-1.157	3.124	1.377	0.647	5.633	2.470
2	-2.616	3.226	1.469	-1.158	2.420	1.080	-2.901	3.085	1.370	1.122	3.803	2.281
3	-1.830	0.951	0.617	-0.434	1.621	0.568	-1.312	1.584	0.669	0.035	2.038	1.072
4	-1.643	1.687	0.892	-1.428	2.421	0.997	-1.780	1.775	0.873	0.393	3.226	1.597
5	-0.838	0.436	0.310	-1.571	1.306	0.571	-1.213	1.047	0.620	0.057	1.840	0.898
6	-1.718	0.609	0.935	-1.318	2.424	0.865	-3.279	2.313	1.306	0.193	3.419	1.825
7	-1.168	1.969	0.799	-1.410	1.496	0.661	-1.035	0.962	0.534	0.167	2.423	1.167
8	-1.980	0.909	0.968	-1.884	1.464	0.765	-2.211	3.341	1.363	0.270	3.532	1.838
9	-0.929	3.341	1.275	-2.114	6.964	2.261	-3.213	5.353	1.940	0.171	7.792	3.241
10	-0.753	0.888	0.372	-0.551	0.545	0.237	-0.283	0.999	0.457	0.073	1.387	0.635
11	-1.855	1.656	1.139	-2.499	1.328	1.157	-2.058	2.522	1.281	0.487	3.232	2.068
12	-1.226	1.632	1.025	-0.916	0.809	0.445	-1.136	1.335	0.763	0.956	1.881	1.353
13	-1.704	1.603	0.761	-2.006	1.355	0.650	-1.349	0.932	0.579	0.455	2.344	1.156
14	-0.846	1.611	0.611	-1.172	1.250	0.696	-0.987	1.224	0.495	0.369	1.993	1.049
15	---	---	---	---	---	---	---	---	---	---	---	---
16	-1.902	1.264	0.876	-1.785	1.341	1.043	-1.575	2.333	0.830	0.913	2.960	1.594
17	-1.075	1.341	0.609	-2.005	2.279	1.175	-1.668	1.249	0.920	0.968	2.429	1.612
18	-0.569	1.124	0.424	-1.289	1.083	0.580	-0.706	0.668	0.417	0.467	1.463	0.831
19	-1.960	1.698	0.948	-1.571	1.183	0.716	-1.916	1.652	1.144	1.011	2.217	1.649
20	-1.292	1.249	0.638	-1.527	0.929	0.437	-0.596	0.552	0.326	0.315	2.014	0.840
21	-0.865	1.293	0.444	-1.566	1.497	0.668	-0.549	0.988	0.357	0.376	1.593	0.877
22	-1.106	2.119	0.879	-2.498	2.125	1.115	-1.990	1.571	1.014	1.019	3.165	1.745
23	-1.801	1.551	0.978	-2.257	1.773	1.162	-1.372	1.968	0.706	0.996	2.659	1.674
24	-1.856	2.157	1.019	-1.039	0.981	0.475	-1.541	1.262	0.721	0.529	2.209	1.336
25	-0.663	1.943	0.637	-0.833	2.100	0.739	-1.110	1.249	0.535	0.247	2.246	1.113
26	-4.620	2.960	2.498	-6.809	5.005	3.401	-6.737	6.192	3.113	1.338	9.090	5.244
27	-2.766	3.447	1.416	-2.924	3.179	1.124	-1.378	2.188	1.035	0.322	3.710	2.083
28	-1.422	2.419	0.926	-1.956	2.251	1.121	-1.903	2.084	0.964	0.784	2.651	1.744
29	-1.218	1.328	0.580	-0.769	2.219	0.812	-1.343	2.282	0.964	0.109	3.161	1.388
30	-0.569	1.381	0.643	-0.861	1.435	0.503	-0.807	1.260	0.518	0.087	1.773	0.966
31	-1.984	1.332	0.866	-1.237	1.464	0.813	-1.394	1.001	0.612	0.944	2.135	1.336
32	---	---	NaN	---	---	NaN	---	---	NaN	---	---	NaN

Table (B-2): GPS broadcast ephemeris accuracy statistics .

(compared with IGS final orbit) [(GPS Lab, 2009)]

Year 2007, Day 110 (20/4/2007)

PRN	X [m]			Y [m]			Z [m]			3D [m]		
	min	max	r.m.s	min	max	r.m.s	min	max	r.m.s	min	Max	r.m.s
1	-1.448	3.783	1.406	-1.533	1.453	0.871	-1.176	2.112	0.908	0.288	4.108	1.887
2	-2.158	2.718	1.330	-1.795	1.381	0.885	-2.793	1.836	1.171	1.034	3.880	1.980
3	-1.569	2.519	1.094	-2.484	0.348	0.984	-3.390	1.794	1.331	0.418	3.722	1.984
4	-1.552	2.392	1.079	-1.861	2.851	1.276	-3.309	1.793	1.111	0.082	4.350	2.007
5	-0.974	1.006	0.538	-1.134	1.177	0.625	-0.540	0.794	0.376	0.143	1.497	0.906
6	-1.458	2.036	0.875	-2.506	1.776	1.038	-3.201	2.780	1.473	0.163	3.454	2.003
7	-1.162	1.872	0.801	-1.981	1.703	0.877	-1.011	0.824	0.367	0.412	2.430	1.243
8	-1.308	1.034	0.567	-0.808	1.130	0.483	-1.251	1.818	0.902	0.340	1.930	1.169
9	-2.197	1.313	0.872	-2.553	1.098	0.710	-1.952	2.585	0.953	0.145	3.333	1.474
10	-0.960	1.766	0.752	-1.324	1.534	0.741	-1.630	1.117	0.585	0.399	2.446	1.207
11	-1.708	1.751	0.960	-1.763	1.418	0.926	-1.545	2.031	1.045	0.579	2.629	1.695
12	-1.233	1.298	0.812	-1.059	1.024	0.539	-1.393	1.600	0.881	1.137	1.684	1.313
13	-2.170	2.919	1.324	-1.892	1.724	1.005	-3.339	2.946	1.660	0.614	3.708	2.349
14	-1.135	1.122	0.597	-2.066	1.415	0.817	-1.663	1.995	0.868	0.472	2.460	1.334
15	---	---	---	---	---	---	---	---	---	---	---	---
16	-1.225	1.323	0.676	-1.454	1.114	0.815	-1.132	1.248	0.578	0.732	1.720	1.206
17	-1.434	1.711	0.801	-2.327	2.324	1.208	-1.921	1.149	0.926	1.102	2.760	1.720
18	-0.939	0.728	0.449	-1.260	1.270	0.754	-1.269	0.991	0.654	0.451	1.864	1.095
19	-1.432	2.510	0.890	-1.230	1.361	0.743	-1.548	1.991	1.031	1.039	2.954	1.551
20	-1.786	1.079	0.716	-1.025	0.613	0.380	-0.985	1.056	0.496	0.295	1.934	0.951
21	-1.260	1.066	0.529	-1.214	1.789	0.762	-0.775	0.923	0.404	0.406	2.041	1.011
22	-1.480	1.439	0.802	-0.867	2.304	0.721	-1.479	1.503	0.944	0.873	2.635	1.433
23	-1.729	1.961	0.982	-2.198	1.620	1.125	-1.280	1.602	0.660	0.921	2.649	1.632
24	-2.135	1.993	1.106	-1.848	0.734	0.702	-1.523	1.409	0.725	0.501	2.449	1.497
25	-2.332	4.165	1.740	-3.085	0.974	1.283	-2.667	3.135	1.429	0.092	4.456	2.591
26	-5.975	2.811	3.104	-8.918	4.099	4.124	-8.815	7.719	3.774	1.439	11.267	6.394
27	-3.256	4.106	1.697	-3.267	3.810	1.311	-1.459	3.034	1.419	0.380	4.383	2.571
28	-0.912	2.976	1.102	-2.589	3.208	1.564	-2.326	2.380	1.105	0.458	3.626	2.209
29	-1.117	2.387	1.133	-1.933	3.243	1.348	-2.091	2.719	1.053	0.586	4.132	2.052
30	-2.319	0.602	0.858	-0.606	1.983	0.599	-1.683	1.123	0.673	0.142	2.479	1.244
31	-1.397	1.279	0.786	-1.425	1.431	0.736	-1.348	1.122	0.677	0.923	1.724	1.272
32	---	---	NaN	---	---	NaN	---	---	NaN	---	---	NaN

Table (B-3): GPS broadcast ephemeris accuracy statistics.

(compared with IGS final orbit) [(GPS Lab, 2009)]

Year 2007, Day 111 (21/4/2007)

PRN	X [m]			Y [m]			Z [m]			3D [m]		
	min	max	r.m.s	min	max	r.m.s	min	max	r.m.s	min	max	r.m.s
1	-6.155	4.567	2.354	-1.658	3.067	1.471	-2.729	3.238	1.790	0.706	6.714	3.303
2	-3.043	3.529	1.579	-1.025	2.688	1.204	-2.896	3.543	1.491	0.997	4.256	2.483
3	-1.039	1.228	0.525	-0.786	1.457	0.667	-1.108	1.446	0.801	0.282	1.730	1.167
4	-1.036	1.105	0.542	-1.066	1.523	0.599	-1.446	0.732	0.575	0.116	1.966	0.992
5	-1.101	0.638	0.439	-0.960	1.060	0.442	-1.190	1.538	0.750	0.145	1.540	0.975
6	-2.318	1.686	1.128	-1.714	2.379	1.130	-3.443	2.675	1.580	0.618	3.762	2.246
7	-2.630	1.127	0.840	-1.161	2.081	0.774	-2.900	2.533	1.200	0.307	3.562	1.657
8	-1.496	0.770	0.645	-0.955	1.223	0.608	-0.995	2.111	0.906	0.541	2.218	1.268
9	-1.879	0.953	0.668	-1.537	1.284	0.491	-1.086	1.166	0.518	0.102	2.190	0.978
10	-0.844	0.763	0.342	-1.025	0.898	0.386	-0.812	1.057	0.475	0.147	1.206	0.701
11	-1.630	1.670	0.896	-1.341	1.474	0.771	-1.673	1.664	0.958	0.591	2.249	1.521
12	-1.799	1.297	0.924	-0.952	1.051	0.609	-1.323	1.706	0.906	1.065	2.180	1.430
13	-1.496	1.314	0.869	-0.924	3.191	0.838	-3.263	1.108	1.044	0.406	3.631	1.596
14	-1.935	1.665	0.910	-2.211	2.381	1.125	-2.384	2.707	1.371	0.981	2.765	1.993
15	---	---	---	---	---	---	---	---	---	---	---	---
16	-0.990	1.469	0.660	-1.519	1.543	0.882	-1.040	1.815	0.710	0.627	2.113	1.311
17	-1.237	1.879	0.858	-2.605	2.805	1.457	-1.926	1.587	1.035	1.073	2.978	1.982
18	-1.290	1.318	0.571	-1.538	1.772	0.898	-1.565	1.160	0.815	0.638	2.169	1.340
19	-1.220	1.199	0.683	-1.056	0.934	0.491	-1.762	1.609	1.082	0.997	1.825	1.371
20	-2.183	1.398	0.886	-1.164	0.815	0.564	-1.073	1.368	0.557	0.394	2.352	1.189
21	-1.343	1.573	0.812	-1.535	1.659	0.961	-1.941	1.638	0.840	0.644	2.304	1.513
22	-2.087	1.644	0.905	-1.582	1.852	0.888	-1.838	2.480	1.321	0.985	2.695	1.830
23	-1.811	1.961	1.038	-2.161	1.801	1.169	-1.169	1.701	0.657	0.991	2.763	1.695
24	-1.463	2.129	0.927	-0.481	0.957	0.272	-1.666	0.217	0.649	0.341	2.359	1.163
25	-1.033	1.901	0.797	-1.568	1.240	0.635	-1.327	1.620	0.721	0.246	2.092	1.248
26	-5.559	2.374	2.692	-7.717	3.692	3.590	-7.688	6.874	3.346	0.743	9.654	5.598
27	-2.702	3.112	1.232	-2.411	3.008	1.035	-1.690	2.146	1.121	0.449	3.570	1.961
28	-0.865	1.319	0.572	-1.374	1.191	0.639	-1.127	1.122	0.444	0.439	1.604	0.966
29	-3.012	4.417	1.581	-2.503	1.135	0.958	-2.875	3.351	1.666	0.187	4.945	2.489
30	-0.941	0.480	0.337	-1.021	0.978	0.451	-0.859	1.073	0.475	0.216	1.247	0.736
31	-1.403	1.606	0.927	-1.880	1.285	0.839	-1.247	1.373	0.630	0.863	2.220	1.400
32	---	---	NaN	---	---	NaN	---	---	NaN	---	---	NaN

Table (B-4): GPS broadcast ephemeris accuracy statistics.
(compared with IGS final orbit)

Year 2007, Day 328 (24/11/2007) [(GPS Lab, 2009)]

PRN	X [m]			Y [m]			Z [m]			3D [m]		
	min	max	r.m.s	min	max	r.m.s	min	max	r.m.s	min	max	r.m.s
1	-4.201	1.168	1.586	-1.576	1.009	0.688	-1.766	2.027	0.975	0.404	4.427	1.984
2	-1.478	1.819	1.040	-1.472	1.631	0.856	-1.316	1.327	0.666	0.921	2.282	1.502
3	-1.605	1.492	0.800	-1.825	1.289	0.912	-2.031	2.359	1.187	0.339	3.116	1.697
4	-1.707	2.741	1.164	-2.067	1.985	0.926	-2.481	2.985	1.311	0.331	3.768	1.983
5	-1.205	1.333	0.675	-0.868	1.853	0.824	-1.239	0.874	0.576	0.217	2.109	1.211
6	-0.751	1.112	0.378	-0.765	0.848	0.385	-1.053	1.210	0.616	0.043	1.397	0.818
7	-2.859	2.646	1.357	-5.736	2.734	1.531	-3.191	3.562	1.930	0.420	6.317	2.812
8	-1.086	1.661	0.546	-1.865	1.954	0.865	-1.840	1.735	0.890	0.238	2.155	1.356
9	-2.130	1.533	0.713	-0.625	1.649	0.583	-2.077	1.502	0.892	0.113	2.857	1.282
10	-2.396	2.789	1.523	-2.473	3.837	1.738	-2.512	2.605	1.413	0.471	4.978	2.708
11	-0.977	1.384	0.564	-1.103	2.009	0.757	-0.948	1.287	0.525	0.308	2.141	1.080
12	-1.043	1.359	0.711	-0.922	1.190	0.641	-1.520	1.478	0.936	1.069	1.739	1.339
13	-0.939	1.783	0.705	-1.072	1.489	0.713	-0.780	0.948	0.372	0.521	2.029	1.070
14	-2.009	2.313	1.151	-1.432	0.903	0.717	-1.263	1.530	0.797	0.346	2.504	1.573
15	-2.943	1.430	1.474	-3.523	1.739	1.320	-4.256	3.093	1.896	0.806	4.594	2.741
16	-1.255	1.566	0.734	-1.195	1.725	0.742	-1.461	1.920	0.877	0.737	2.258	1.364
17	-1.489	1.585	1.071	-1.604	1.226	0.798	-1.301	1.516	0.842	1.331	2.089	1.579
18	-1.579	1.197	0.837	-0.621	0.936	0.424	-1.589	1.350	0.644	0.402	1.623	1.138
19	-1.411	1.397	0.777	-1.706	1.542	0.981	-1.444	2.355	1.036	1.190	2.448	1.624
20	-1.188	1.310	0.734	-1.448	1.295	0.799	-0.867	1.374	0.584	0.518	1.923	1.232
21	-1.807	2.204	0.950	-1.457	0.944	0.623	-2.410	2.477	1.277	0.898	2.715	1.709
22	-0.962	1.274	0.596	-1.354	1.632	0.962	-1.168	1.926	0.901	0.975	1.977	1.447
23	-1.933	1.697	1.091	-1.533	1.409	0.901	-1.634	1.759	0.983	0.933	2.480	1.723
24	-1.680	1.317	0.713	-2.702	1.139	1.191	-1.132	3.001	1.432	0.356	3.590	1.994
25	-1.533	1.553	0.733	-1.108	1.646	0.713	-1.226	1.454	0.674	0.209	2.249	1.225
26	-1.708	1.104	0.594	-0.933	1.104	0.482	-2.317	1.631	0.879	0.030	2.419	1.166
27	-3.109	1.905	0.940	-2.299	2.629	1.135	-1.947	1.106	0.813	0.179	4.191	1.683
28	-1.238	2.126	0.949	-1.932	1.173	0.934	-0.841	0.865	0.418	0.398	2.216	1.396
29	---	---	---	---	---	---	---	---	---	---	---	---
30	-0.966	1.151	0.580	-0.598	1.266	0.480	-0.824	1.219	0.538	0.430	1.478	0.925
31	-1.200	1.144	0.601	-1.762	1.066	0.737	-1.302	1.316	0.812	0.892	1.772	1.250
32	---	---	NaN	---	---	NaN	---	---	NaN	---	---	NaN

Table (B-5): Satellite IDs for Static GPS (elevation angle 15°).

[GNSS Solution (2007)]

Visible Satellite ID	Base point	2.37 km			4.05 km			5.62 km			
	19/4/2007	20/4/2007									
		10 min	15 min	20 min	10 min	15 min	20 min	10 min	15 min	20 min	30 min
1	√	√	√	√	√	√	√			√	√
2											
3		√	√	√	√	√	√	√	√	√	√
4											
5											
6											
7								√	√		
8											
9											
10											
11	√		√	√							
12											
13	√										
14	√	√	√	√	√	√	√	√	√	√	√
15											
16					√	√	√	√	√	√	√
17											
18					√	√	√	√	√	√	√
19	√	√	√	√	√	√	√	√	√	√	√
20	√			√							
21					√	√		√	√	√	√
22		√	√	√	√	√	√	√	√	√	√
23	√										
24											
25	√										
26											
27											
28											
29											
30											
31	√		√	√							

Table (B-6): Satellite IDs for Static GPS (elevation angle 15°).

[GNSS Solution (2007)]

Visible Satellite ID	8.71 km					12.52 km					
	20/4/2007										
	10 min	15 min	20 min	30 min	45 min	10 min	15 min	20 min	30 min	45 min	60 min
1			√	√	√					√	√
2											
3	√	√	√	√	√	√	√	√	√	√	√
4											
5											
6	√	√				√	√				
7	√	√	√	√		√	√	√	√		
8											
9											
10											
11				√	√						√
12											
13											
14	√	√	√	√	√	√	√	√	√	√	√
15											
16	√	√	√	√	√	√	√	√	√	√	
17											
18	√	√	√	√	√	√	√	√	√	√	√
19	√	√	√	√	√			√	√	√	√
20											√
21	√	√	√	√	√	√	√	√	√	√	
22	√	√	√	√	√	√	√	√	√	√	√
23											
24											
25											
26											
27											
28											
29											
30											
31											√

Table (B-7): Satellite IDs for Static GPS (elevation angle 15°).

[GNSS Solution (2007)]

Visible Satellite ID	15.02 km						16.25 km			
	20/4/2007						21/4/2007			
	10 min	15 min	20 min	30 min	45 min	60 min	10 min	45 min	60 min	75 min
1					√	√		√	√	√
2										
3	√	√	√	√	√	√	√	√	√	√
4										
5										
6	√	√	√				√			
7	√	√	√	√			√			
8										
9										
10										
11						√			√	√
12										
13										
14		√	√	√	√	√		√	√	√
15										
16	√	√	√	√	√	√	√	√		
17										
18	√	√	√	√	√	√	√	√		
19				√	√	√		√	√	√
20									√	√
21	√	√	√	√	√		√	√		
22	√	√	√	√	√	√	√	√	√	√
23										√
24										
25										
26										
27										
28										
29										
30										
31									√	√

Table (B-8): Satellite IDs for Static GPS (elevation angle 15°).

[GNSS Solution (2007)]

Visible Satellite ID	18.80 km							31 km				
	21/4/2007							24/4/2007				
	10 min	15 min	20 min	30 min	45 min	60 min	75 min	15 min	30 min	60 min	90 min	120 min
1			√	√	√	√	√					
2												√
3	√	√	√	√	√	√	√	√	√	√		
4												
5										√	√	√
6	√	√						√	√	√	√	√
7	√	√	√	√				√	√	√	√	√
8												
9												√
10											√	√
11						√	√					
12											√	√
13												
14	√	√	√	√	√	√	√	√	√			
15									√	√		
16	√	√	√	√	√			√	√	√	√	
17												
18	√	√	√	√	√			√	√	√	√	√
19				√	√	√	√					
20						√	√					
21	√	√	√	√	√			√	√	√	√	√
22	√	√	√	√	√	√	√	√	√	√	√	
23							√					
24									√	√	√	√
25							√					
26												
27												
28												
29												
30										√	√	√
31										√	√	√

<u>Note for the base line length of 31 kms</u>

The original raw data file for elapsed time of 90 min had a problem where the processing data of this file as follow in table (B-9)

Table (B-9): Processing data of 31 km base line before deselect Satellite No.7.

[GNSS Solution (2007)]

Type	Elapsed Time	Vector Length (L) in m	Coordinates UTM Zone 36 N East in m	North In m	Ellip.H in m	Vector Rel.Error in m (95% Confidence)	Sat. No.	PDOP
Stand alone	15 min	-	492727.100	2699349.000	102.460	-	-	-
	30 min	-	492724.333	2699347.433	106.885	-	-	-
	60 min	-	492723.326	2699346.401	103.220	-	-	-
	90 min	-	492722.619	2699346.309	103.501	-	-	-
	120 min	-	492725.617	2699346.876	99.828	-	-	-
Broadcast	15 min	31068.804	492724.344	2699347.147	101.116	0.297	8.0	2.0
	30 min	31068.862	492724.496	2699347.191	101.263	0.150	9.0	1.9
	60 min	31068.920	492724.486	2699347.250	101.336	0.150	10.0	1.4
	<u>90 min</u>	*<u>31068.912</u>*	*<u>492724.454</u>*	*<u>2699347.245</u>*	*<u>101.479</u>*	*<u>0.295</u>*	*<u>11.0</u>*	*<u>1.5</u>*
	120 min	31068.865	492724.449	2699347.199	101.297	0.150	11.0	1.4
Precise final IGS	15 min	31068.800	492724.340	2699347.143	101.113	0.297	8.0	2.0
	30 min	31068.865	492724.493	2699347.195	101.268	0.150	9.0	1.9
	60 min	31068.918	492724.484	2699347.248	101.343	0.150	10.0	1.4
	<u>90 min</u>	*<u>31068.912</u>*	*<u>492724.447</u>*	*<u>2699347.246</u>*	*<u>101.48</u>*	*<u>0.295</u>*	*<u>11.0</u>*	*<u>1.5</u>*
	120 min	31068.864	492724.446	2699347.198	101.296	0.150	11.0	1.4

From table (B-9); the vector relative error and coordinates had irregular values in the sequence of another elapsed time so from the GPS broadcast ephemeris accuracy statistics (compared with IGS final orbit) as shown in table (B-4); the maximum 3D value of satellite No. 7 is out of range (6.317m).

To solve this problem; satellite No. 7 was deselected in GNSS Solution Software.

The processing data after deselect the satellite No.7 as shown in table (B-10)

Table (B-10): Processing data of 31 km base line after deselect Satellite No.7.

[GNSS Solution (2007)]

Type	Elapsed Time	Vector Length (L) in m	Coordinates UTM Zone 36 N East in m	North In m	Ellip.H in m	Vector Rel.Error in m (95% Confidence)	Sat. No.	PDOP
Stand alone	15 min	-	492727.100	2699349.000	102.460	-	-	-
	30 min	-	492724.333	2699347.433	106.885	-	-	-
	60 min	-	492723.326	2699346.401	103.220	-	-	-
	90 min	-	492722.619	2699346.309	103.501	-	-	-
	120 min	-	492725.617	2699346.876	99.828	-	-	-
Broadcast	15 min	31068.804	492724.344	2699347.147	101.116	0.297	8.0	2.0
	30 min	31068.862	492724.496	2699347.191	101.263	0.150	9.0	1.9
	60 min	31068.920	492724.486	2699347.250	101.336	0.150	10.0	1.4
	90 min	*31068.865*	*492724.495*	*2699347.194*	*101.524*	*0.150*	*9*	*1.8*
	120 min	31068.865	492724.449	2699347.199	101.297	0.150	11.0	1.4
Precise final IGS	15 min	31068.800	492724.340	2699347.143	101.113	0.297	8.0	2.0
	30 min	31068.865	492724.493	2699347.195	101.268	0.150	9.0	1.9
	60 min	31068.918	492724.484	2699347.248	101.343	0.150	10.0	1.4
	90 min	*31068.864*	*492724.495*	*2699347.194*	*101.524*	*0.150*	*9*	*1.8*
	120 min	31068.864	492724.446	2699347.198	101.296	0.150	11.0	1.4

Appendix C

Field Experimental Work Gallery

1. **<u>Static GPS Experimental Work:</u>**

Fig. (C-1): Aswan Cable Stayed Bridge Iterance of Aswan- Edfo Desert Road.

Fig. (C-2): Base Point Setup.

Fig. (C-3): Rover Point Setup.

Fig. (C-4): Promark 3.0 Screen (data collect).

Fig. (C-5): Work group (1) in field.

Fig. (C-6): Work group (2) in field.

Fig. (C-7): Work group (3) in field.

2. <u>Kinematic GPS Technique :</u>

Fig. (C-8): Initialization by bar in base point .

Fig. (C-9): Set up of Rover Receiver Pole for Kinematic Technique .

Fig. (C-10): Rover Receiver Pole for Kinematic Technique .

Fig. (C-11): Work Group for DEM .

دراسة تقييم دقة استخدام (النظام الامريكي لتوقيع الاحداثيات باستخدام الاقمار الصناعية) في التطبيقات المساحية في جنوب مصر

يعد نظام الرصد المساحى باستخدام الاقمار الصناعية من أحدث طرق الرصد في العالم لذا كان الواجب علينا أن ندرس مميزات استخدام هذا النظام في منطقة جنوب مصر (محافظة أسوان كمثال)، وذلك لاستخدام هذا النظام في عمليات التطوير والتنمية في اقليم جنوب الصعيد.

ونظراً لتوجه الحكومة المصرية لزيادة خطط التنمية في منطقة جنوب الصعيد والتي تتضمن العديد من المشروعات التطويرية والتي بمقتضاها تساعد علي الخروج من الوادي الضيق حول نهر النيل الي المساحات الصحراوية الشاسعة وخاصة حول بحيرة ناصر خلف السد العالي.

وهنا تبرز أهمية وجود دراسة تطبيقية لدراسة دقة تقييم نظام GPS في التطبيقات المساحية في جنوب مصر.

هذا البحث يتكون من جزئين من التجارب العملية:

- الجزء الاول من التجارب العملية هو دراسة استخدام نظام الرصد الثابت باستخدام الاقمار الصناعية (Static GPS Technique) في قياس أطوال خطوط قاعدة تصل حتي 20 كم وتم اضافة قياس خط قاعدة طوله 31 كم وقد اوضحت التجارب أن مقدار الخطأ النسبي يساوي 5 جزء في المليون لكل كيلو متر طولي عند عدد أقمار صناعية مرئية لا تقل عن 6 أقمار و قيمة PDOP لا تزيد عن 2.70.

- الجزء الثاني من العملي يتضمن جزئين

هو مقارنة النموذج الرقمي لإرتفاعات سطح الأرضDEM ما بين Total Station كقيم مرجعية والقيم الناتجة من نظام الرصد المتحرك باستخدام الاقمار الصناعية (Stop and Go and Kinematic GPS Technique) و أوضحت التجارب أن قيم RMSE في حالة

Stop and Go تساوي 9.736 سم بينما قيمة RMSE في حالة Kinematic تساوي 12.05 سم.

- رصد طريق طولي باستخدام Total Station and Kinematic واوضحت النتايج أن أن قيم التغير تتراوح ما بين 12.93 سم و 13.76-سم وأن قيمة RMSE تساوي 5.99 سم وهذه الدقة تناسب وتساعد في عمل الخرائط الكنتورية والميزانيات الشبكية حول بحيرة ناصر لأستصلاح الاراضي الشاسعة حولها.

كليــــــة الهندســــــــة بأسوان
قسم الهندسة المدنية

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رسالة ماجستير

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جامعة جنوب الوادي

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