PALESTINE POLYTECHNIC UNIVERSITY

COLLEGE OF ENGINEERING & TECHNOLOGY

CIVIL ENGINEERING DEPARTMENT

SURVEYING ENGINEERING

GRADUATION PROJECT

COMPARISON BETWEEN SURVEYING TECHNIQUES STATIC AND REAL TIME KINEMATIC USING GPS

PROJECT TEAM

MUFEED AL- SHAMISTI

PROJECT SUPERVISOR
ENG. FAYDI SHABANEH

HEBRON-PALESTINE
DECEMBER-2005

CERTIFICATION

PALESTINE POLYTECHNIC UNIVERSITY

HEBRON-PALESTINE

COMPARISON BETWEEN SURVEYING TECHNIQUES STATIC & REAL TIME KINEMATIC USING GPS

MUFEED AL- SHAMISTI

By the guidance of our supervisor, and the approval of member of the testing Committee ,this project is delivered to the Department of Civil and Architectural Engineering , in the College of Engineering and Technology to be as partial fulfillment of the requirements of the department for the degree of B.Sc in Surveying and Geomatics Engineering

Supervisor signature: Head of Dep. signature

Name:……………… Name:……………….

Committee member's signature

Name:……………… Name:……………… Name:………………

HEBRON-PALESTINE

DECEMBER-2005

DEDICATION

الاهداء

إلى روح المرحوم الأستاذ" كمال غطاشه"......

إلى والدتي ......

إلى والدي ......

إلى شهدائنا الأبرار...

إلى كل من امن بفكرته وسعى لأجلها بكل جهد وفكر.

إلى أحبائنا الذين تتوق أنفسهم للتفوق والنجاح

اهدي هذا الجهد المتواضع راجين المولى أن ينال الرضا.

م.مفـــــيد الـــــشـــــامـــــســـــطي

ACKNOWLEDGMENT

شكر وتقدير

إن الشكر والمنة لله و حده دائما الذي لايحمد على مكروهسواه.

نتقدم بأرفع وأسمى آيات الشكر والامتنان والتقدير إلى جامعة بوليتكنك فلسطين....

إلى دائرة الهندسة " خاصة دائرة الهندسة المدنيه....

إلى كل من قدم لنا مساعدة في إنجاز هذا المشروع....

إلى نهر العطاء الدائم م. فيضي شبانه

الذي قدم لي العون والمشورة في كل صغيره و كبيرة

وجميع الهيئه التدرسيه في الجامعه.

م.مفـــــيد الـــشـــامـــســـــطي

ABSTRACT

COMPARISON BETWEEN SURVEYING TECHNIQUES STATIC AND REAL TIME KINEMATIC USING GPS

Project Team
Mufeed AL- Shamisti

Project Supervisor
Eng.Faydi shabaneh

Comparison between different GPS observation techniques regarding accuracy. The main comparisons in this project are accuracy comparisons between two different measuring methods with GPS in different time epochs:

- Real Time Kinematic (RTK).
- Static.

The accuracy and precision of the different methods was controlled with a large amount of measuring data.

This has been accomplished; by first carefully determine the position of a GPS antenna, permanently placed on the roof of the PPU University in Hebron. This was done by simultaneous, static observations from two well-positioned control points A& B.

After this, four control points, placed in the nearby region of the PPU University were chosen. The positioning of the roof antenna was earlier very well decided.
These network points were observed with each GPS observation techniques method in different time epochs and the results were then compared. The comparison shows that there is significant difference in how accurate each method is measuring .

الملخـــــــــــــــــــــص

COMPARISON BETWEEN SURVEYING TECHNIQUES STATIC AND REAL TIME KINEMATIC USING GPS

أعداد

مفيد الشامسطي

اشرف

المهندس فيضي شبانه

يعتبر نظام تحد يد المواقع بالأقمار الصناعية(GPS) من احدث أنظمة التكنولوجيا المعاصرة والتي أحدثت تطوراً في معظم التطبيقات بتكامله مع العلوم الأخرى.
حيث تم في المشروع المقارنة بين القراءات باستخدام جهاز(GPS) بالنسبة الدقة القراءات، حيــث أن المقارنة الرئسيه في هذا المشروع هي مقارنة الدقة بين طرقتي قياس باستخدام جهازGPS في أوقات مختلفه وهما طريقة RTK و STATIC.

ولقد تم ضبط ألدقه بين الطرقتين عن طريق الحصول على بيانات حقليه كثيرة،عن طريق وضع لاقط هوائي والذي يتم تثبيته بشكل دائم فوق جامعة بوليتكنك فلسطين في الخليل،وتثبيت النقاط المرجعيه(A,B) بطريقة (STATIC) بشكل دقيق.

وتمثل المشروع بشكل أساسي برصد أربعة نقاط تحكم قريبه من جامعة بوليتكنك فلسطين،وتم جمع المعلومات لهذه الشبكة في أوقات مختلفه و مقارنة النتائج، وتشير المقارنة على انه يوجد فرق في دقة كل طريقة قياس من الطرق ألسابقه، حيث تبين أن طريقة STATIC أعطت نتائج أدق من الطريقه الأخرى RTK بناء على الانحراف المعياري لبيانات الطريقتين .

TABLE OF CONTENTS

Chapter 3 **GPS Satellite Surveying**

Chapter 4 **Software and 5700 GPS Receiver**

List Of Tables

List Of Figures

CHAPTER ONE

INTRODUCTION

This chapter includes introduction of Global Positioning System "GPS".

1.1 Background

1.2 Overview of GPS

1.3 Project importance

1.4 Project objectives

1.5 Study area

1.6 Equipment necessary to carry out a GPS survey

1.7 Project out line

1.8 Problems

1.9 Previous studies

1.10 Time planning

CHAPTER ONE

INTRODUCTION

1-1 Background

The aim of this project is to study the fundamentals of the Global Positioning System and other systems used in the same area, the error sources and some of techniques that can be used to get higher accuracies, which is required in most of the surveying projects.

The project is divided in to two parts; the first part describes the GPS system and the principles of operation. In the second part of the project I will try to describe the field and office procedures of surveys, planning GPS surveys. Performing GPS surveys and the future of GPS in the practice of surveying.

1-2 Overviews of GPS

The Global Positioning System (GPS) is a satellite-based positioning system. It is a complex system that can be used to achieve position accuracies ranging from 100 m to a few millimeters depending on the equipments used and procedures followed.

GPS is the best system today able to determine your position on the Earth anytime, in any weather, anywhere (between the latitudes of 80˚N and 80˚S). The satellites transmit signals that can be detected by anyone with a GPS receiver. Using the receiver, you can determine your location with great precision.

1-3 Project importance

Importance of using GPS in Surveying

- Surveying and mapping, including cadastral and urban networks, data capture surveys for Geographic Information Systems (GIS), engineering surveys, photogrammetrical control (airborne and terrestrial), and geophysical resource surveys.
- Geodetic applications, including the establishment of control networks over regional and continental extent, height and geoid determination, precise engineering and subsidence monitoring surveys.
- Geodynamic applications, for measuring the relative position of a regional network at regular intervals in order to study horizontal and vertical crustal motion.
- Land navigation, to support emergency vehicles (police, search & rescue, etc.) and for the monitoring of cars, taxis, dangerous and valuable cargoes, trucks and railways.
- Transportation and communication, to support aids for navigation for land, sea and air users, land operations taking advantage of permanent GPS stations, time transfer operations, etc.
- Recreational uses, for hiking, orienteering, etc.

1-4 Project objectives

- Get enough training on the new receiver in PPU Trimble 5700.
- Comparison between surveying techniques using GPS.
- Check coordinates in one or more national grid points.
- Support map control in photogrammetry.
- Establishing geodetic control in PPU Campus.
- Supporting engineering construction

1-5 Study Area

Wadi AL Hariah (PPU campus) in the southern part of Hebron city was chosen as a pilot area, figure1.1.

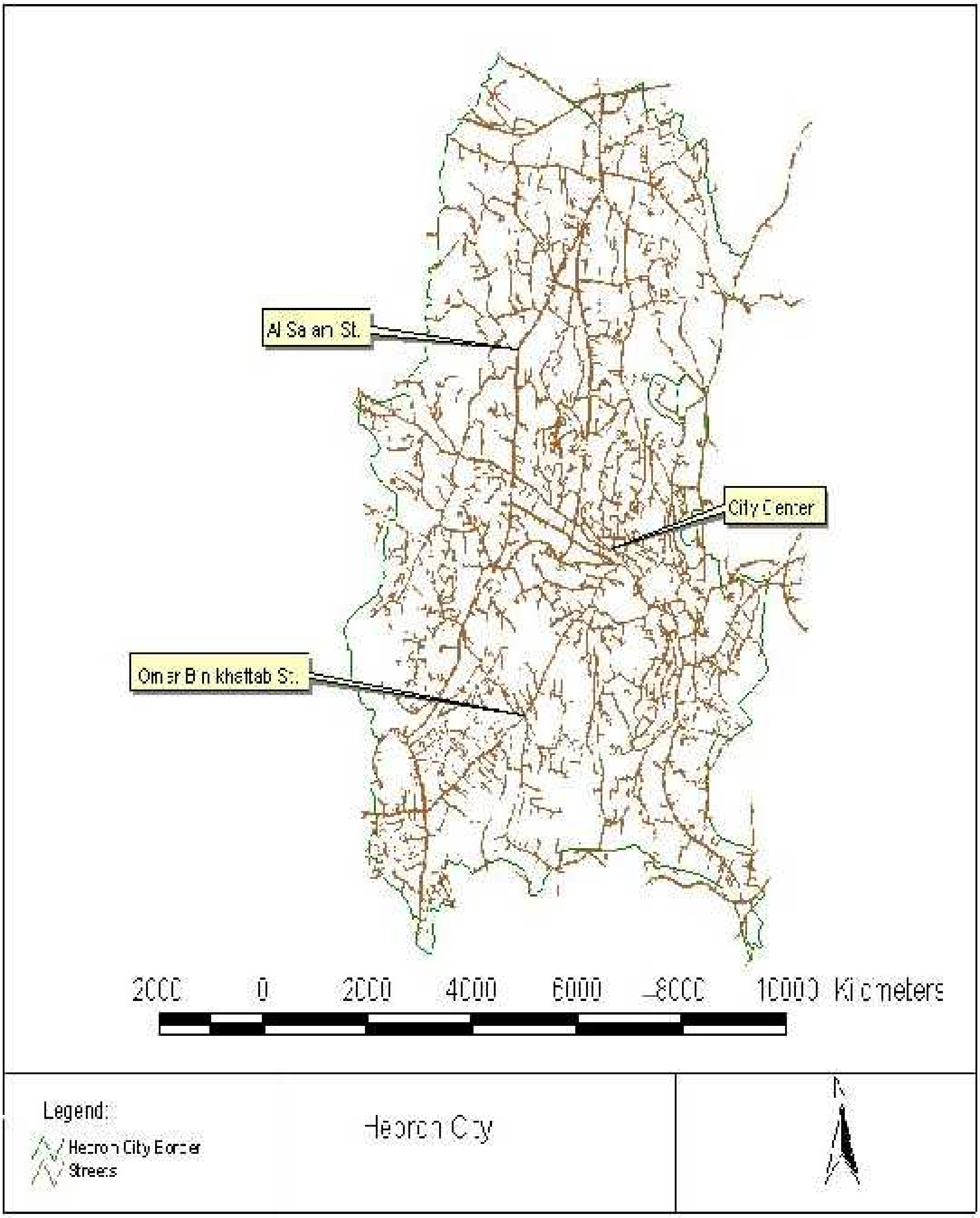


Figure 1.1: Hebron City (source: GIS Unit in Hebron Municipality)

1-6 Equipment necessary to carry out a GPS survey

Equipment for Instrument Station

The following list may be taken as a guide:

- GPS receiver, antenna and associated cabling.
- External batteries (including spares), battery charger.
- Data storage consumerables, for example memory cards, diskettes, and possibly a P.C. computer for logging or downloading data.
- Antenna tripod, tribrachs or adaptors for mounting antenna on pillars.
- Pocket tape, 30m tape, plumbobs, umbrella and supports, etc.
- Field book(s), maps, access details, observation schedule, instructions.
- Useful ancillary equipment: camera, watch, communications equipment.
- Transport vehicle.

Equipment at Base Station or Field Office

The following may be taken as a guide:

- Portable computer with suite of software for the downloading, checking, pre-processing, of GPS data collected by individual field parties if available.
- List of station coordinates topographic maps, observation schedules, recovery access diagrams, client instructions, useful contact addresses and telephone numbers.
- Data storage, for example diskettes, for the archiving and storage of tracking data.
- Cables and ancillary equipment for downloading data from GPS receivers.
- Computer modem for transmitting data to head office.
- Communication equipment, for example radios, or at the very least mobile telephone to ensure contact between field parties and head office.

- Spare GPS receivers, cables, batteries, and other field equipment that may not be needed by field parties every day such as, for example, theodolites, Electronic Distance Measurement (EDM), etc.
- Transportation.

1-7 Project out line

1. Introduction
2. Theoretical background
3. GPS Satellite surveying
4. Software and 5700 GPS receiver.
5. Field Measurements.
6. Calculation.
7. Conclusion and recommendations.

1-8 Problems

There are many Problems bid in this project due to some reasons, these Problems can be summaries as follow:

- Lack of references, the reference that covers this project does not exist.

- Limitation of using GPS receiver because of its higher price and political situation (impossible to use it out PPU).

1-9 Previous studies

No previous studies deal with the project topics found in the country, but out side there are many project using different methods.

1.10Time planning

This project lasts 32 Weeks divided into nine tasks, explained in table 1.1.

<u>Weeks</u>

Project phases	1		3		5		7		9		11		13		15		17		19		21		23		25		27		29		31	
Problem definition																																
Literature Review																																
Training Trimble Geomatis Office																																
Training GPS																																
Collecting Data																																
Determining Standard deviation																																
Writing and edit report																																
Final Project and presentation																																
documentation																																

Table 1.1: Time planning

CHAPTR TWO

THEORETICAL BACKGROUND

This chapter includes theoretical background of Global Positioning System "GPS".

2.1 Global Positioning System "GPS".

2.2 NVSTAR System Configuration.

2.3 The GPS Satellite Signal.

2.4 GPS Error Sources.

2.5 The Impact of Satellite Geometry.

2.6 How GPS Works.

2.7 Other System (GALILEO and GLONASS).

CHAPTER TWO

THEORETICAL BACKGROUND

2.1 Global Positioning System "GPS"

2.1.1 What Is GPS?

The Global Positioning System (GPS) is a space-based, microwave, 24-hour, all-weather, global military navigation system designed, deployed, financed and managed by the U.S. military authorities. Since GPS was declared operational in mid-1993 it has had a profound impact on the art and practice of most forms of positioning and navigation. However, GPS has already a tremendous impact on surveying, initially as a technology for "control surveys", for which purpose it was first introduced into many countries during the early 1980’s, well before the full satellite constellation was available to navigation users. In fact, the application of GPS for control surveys (or so-called "geodetic" surveys) was the first civilian use of GPS which was well beyond that for which GPS was originally intended by its military designers.

Nowadays GPS surveying techniques have completely replaced technologies such as Doppler satellite positioning and even long-range Electronic Distance Measurement (EDM) for traditional first (and lower) order control surveys. However, the adoption of GPS is not restricted to control survey applications. More and more GPS is being used for cadastral, topographical and engineering surveying. The constraints that have previously restricted the application of GPS technologies, primarily those of cost (capital and running) and productivity (measured in terms of the numbers of points which can be coordinated in a day) are being aggressively addressed by the manufacturers, and it is confidently predicted that GPS will shortly be used by the majority of surveyors and geomatic engineers.

2.1.2 GPS Program History

Since the early 1960s various U.S. agencies have had navigation satellite programs. The John Hopkins' Applied Research Laboratory sponsored the TRANSIT program and the U.S. Navy (USN) sponsored the TIMATION (TIMe navigATION) program. TIMATION was a program to advance the state of the art for two-dimensional (latitude and longitude) navigation. TRANSIT became operational in 1964 and is currently providing navigation service to low dynamic vehicles such as ships. It is scheduled to be phased out in 1996. The U.S conducted concept studies to assess a three-dimensional (latitude, longitude, and altitude) navigation system called 621B.

The first generation of satellites launched between 1978and 1985 were the Block I (research and development).The second series launches(the Block II or production satellites) was begun in 1989.The GPS constellation was declare fully operation in 1995(prior to this time ,GPS positioning was intermittent due to lack of full coverage).Lunching of Block IIR (R is for replenishment) satellites began in 1997 an is still underway .Future launches of a Block IIF (Follow-on)series ,along with elated GPS modernization initiatives (i.e.GPS III), will keep the system operational for atleast the next two decades. The actual number of operational satellites and their location varies at any given time as satellites are constantly being replaced, realigned and upgraded see Table2-1 [*].

NAVSTAR GPS is not the only global navigation satellites system (GNSS).Russia maintains a similar global orbiting satellites system (GLONASS) of nominally 24 satellites. Some high-end receivers can acquire and process both the GPS and GLONASS satellites simultaneously .This capability will be further expanded when the process European Union 30- satellites navigation system (GALILEO) is implemented in a decade or so. Japan and China are also considering development of their own (GNSS).

---

[*] Source :US Coast Guard Navigation Center (www.navcen.uscg.gov)

Table 2.1: Satellites Constellation Status

SVN No	PRN No	Launch Date	Block-Mission No	Operational Date	Months Operat`al	Years Operator
1	4	I-1	22-Feb-78	29-Mar-78	21.9	1.825000
2	7	I-2	13-May-78	14-Jul-78	25.5	2.125000
3	6	I-3	06-Oct-78	09-Nov-78	161.3	13.441667
4	8	I-4	11-Dec-78	08-Jan-79	93.6	7.800000
5	5	I-5	09-Feb-80	27-Feb-80	45	3.750000
6	9	I-6	26-Apr-80	16-May-80	126.8	10.566667
7	**	I-7	18-Dec-81	**	0	0.000000
8	11	I-8	14-Jul-83	10-Aug-83	116.8	9.733333
9	13	I-9	13-Jun-84	19-Jul-84	115.2	9.600000
10	12	I-10	08-Sep-84	03-Oct-84	133.5	11.125000
11	3	I-11	09-Oct-85	30-Oct-85	99.9	11.733330
14	14	II-1	14-Feb-89	14-Apr-89	141.4	11.783333
13	2	II-2	10-Jun-89	12-Jul-89	138.6	11.550000
16	16	II-3	17-Aug-89	13-Sep-89	136.4	11.66667
19	19	II-4	21-Oct-89	14-Nov-89	134.5	11.208333
17	17	II-5	11-Dec-89	11-Jan-90	132.6	11.050000
18	18	II-6	24-Jan-90	14-Feb-90	127.5	10.625000
20	20	II-7	25-Mar-90	19-Apr-90	72.7	6.058333
21	21	II-8	02-Aug-90	31-Aug-90	125	10.416667
15	15	II-9	01-Oct-90	20-Oct-90	123.3	10.275000
23	23	II-10	26-Nov-90	10-Dec-90	121.6	10.133333
24	24	II-11	03-Jul-91	30-Aug-91	113	9.416667
25	25	II-12	23-Feb-92	24-Mar-92	106.2	8.850000
28	28	II-13	09-Apr-92	25-Apr-92	101.1	8.425000
26	26	II-14	07-Jul-92	23-Jul-92	102.2	8.516667
27	27	II-15	09-Sep-92	30-Sep-92	100	8.333333

32	1	II-16	22-Nov-92	11-Dec-92	97.6	8.133333
29	29	II-17	18-Des-92	05-Jan-93	96.8	8.066667
22	22	II-18	02-Feb-93	04-Apr-93	93.8	7.816667
31	31	II-19	30-Mar-93	13-Apr-93	93.5	7.791667
37	7	II-20	13-May-93	12-Jun-93	91.6	7.633333
39	9	II-21	26-Jun-93	21-Jul-93	90.3	7.525000
35	5	II-22	30-Aug-93	20-Sep-93	88.3	7.358333
34	4	II-23	26-Oct-93	01-Dec-93	85.9	7.158333
36	6	II-24	10-Mar-94	28-Mar-94	82	6.833333
33	3	II-25	28-Mar-96	09-Apr-96	57.7	4.808333
40	10	II-26	16-Jul-96	15-Aug-96	53.5	4.45333
30	30	II-27	12-Sep-96	01-Oct-96	51.9	4.325000
38	8	II-28	06-Nov-97	18-Des-97	37.4	3.116667
42	12	IIR-1	17-Jan-97	**	0	0.000000
43	13	IIR-2	22-Jul-97	31-Jan-98	36	3.000000
46	11	IIR-3	06-Oct-99	03-Jan-00	12.9	1.075000
51	20	IIR-4	10-May-00	01-Jun-00	7.9	0.658333
44	28	IIR-5	16-Jul-00	17-Aug-00	5.4	0.450000
41	14	IIR-6	10-Nov-00	10-Dec-00	1.6	0.133333
54	18	IIR-7	30-Jan-01	15-Feb-01	-----	----

2.2 NVSTAR System Configuration

The NVSTAR GPS consists of three distinct segments, Figure 2.1

1 -The Space segments (satellites)

2 -The Control segments (ground tracking and monitoring stations)

3- The User segments (air, land, and sea-based receivers).

.

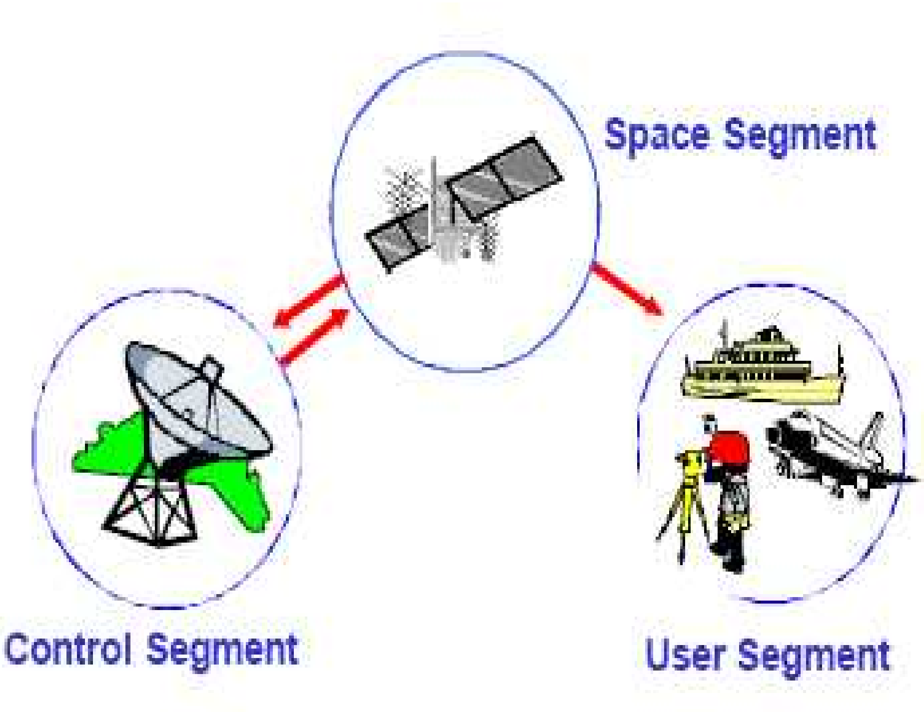


Figure 2.1: NAVSTAR GPS Major Segments[1]

2.2.1 Space Segment

The GPS Space Segment consists of 24 NAVSTAR satellites. The satellites are arranged in six orbital planes with four satellites in each plane the orbital planes have an inclination angle of 55 degrees relative to the earth's equator. The satellites have an average orbit altitude of 20200 kilometers above the surface of the earth. Figure 2-2 illustrates the GPS satellite constellation.

The satellites complete one orbit in approximately 11 hours and 58 minutes. Since the earth is rotating under the satellites, the satellites trace a track over the earths surface which repeats every23 hours and 56 minutes. A user at a fixed location on the ground will observe the same satellite each day passing through the same track in the sky, but the satellite will raise and set four minutes earlier each day, due to the 4 minute

[1] From reference No. 8

difference between the rotational period of the earth and two orbital periods of a satellite.

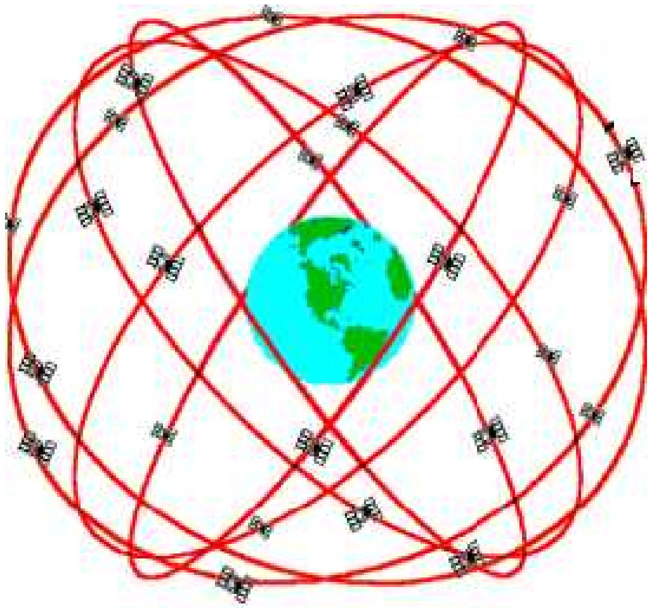


Figure 2.2: GPS Satellite Constellation[1]

2.2.2 Control Segment

The Control Segment primarily consists of a Master Control Station (MCS), at Falcon Air Force Base (AFB) in Colorado Springs, USA, plus monitor stations (MS) and ground antenas (GA) at various locations around the world. The monitor stations are located at Falcon AFB, Hawaii, Kwajalein, Diego Garcia, and Ascension figure 2.3. All monitor stations except Hawaii and Falcon AFB are also equipped with ground antennas.

[1] From reference No. 8

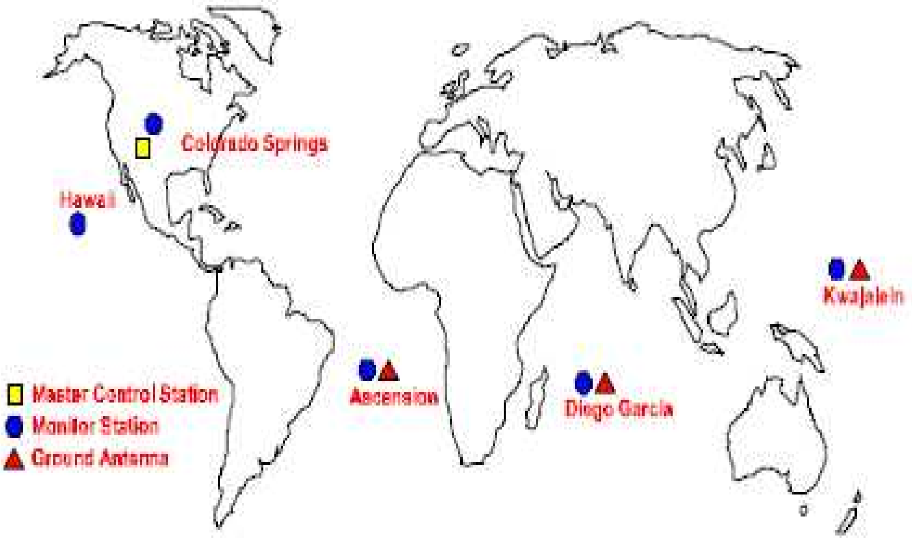


Figure 2.3: Master Control Station.[1]

The MCS is the central processing facility for the Control Segment and is responsible for monitoring and managing the satellite constellation. The MCS functions include control of satellite station-keeping maneuvers, reconfiguration of redundant satellite equipment, regularly updating the navigation messages transmitted by the satellites, and various other satellite health monitoring and maintenance activities.

The monitor stations passively track all GPS satellites in view, collecting ranging data from each satellite. This information is transmitted to the MCS where the satellite ephemeris and clock parameters are estimated and predicted. The MCS uses the ground antennas to periodically upload the ephemeris and clock data to each satellite for retransmission in the navigation message. Communications between the MCS the MS and GA are typically accomplished via the U.S. Defense Satellite Communication System (DSCS). The navigation message update function is graphically depicted in Figure 2.4

[1] From reference No. 8

Figure 2.4: Monitor Station and Ground Antenna[1]

2.2.3 User Segment

The User Segment consists of receivers specifically designed to receive, decode, and process the GPS satellite signals. Receivers can be stand-alone, integrated with or embedded into other systems. GPS receivers can vary significantly in design and function, depending on their application for navigation, accurate positioning, time transfer, surveying and attitude reference.

2.3 The GPS Satellite Signal

Each GPS satellite transmits a unique navigational signal centered on two L-band frequencies of the electromagnetic spectrum: L1 at 1575.42MHz and L2 at 1227.60MHz. At these microwave frequencies the signals are highly directional

[1] From reference No. 8

and hence are easily blocked, as well as reflected, by solid objects and water surfaces. However, clouds are easily penetrated, but the signals can be blocked by dense or wet foliage.

The satellite signals basically consist of (see Figure 2.5):

- The two L-band carrier waves.
- The ranging codes modulated on the carrier waves.
- The Navigation Message.

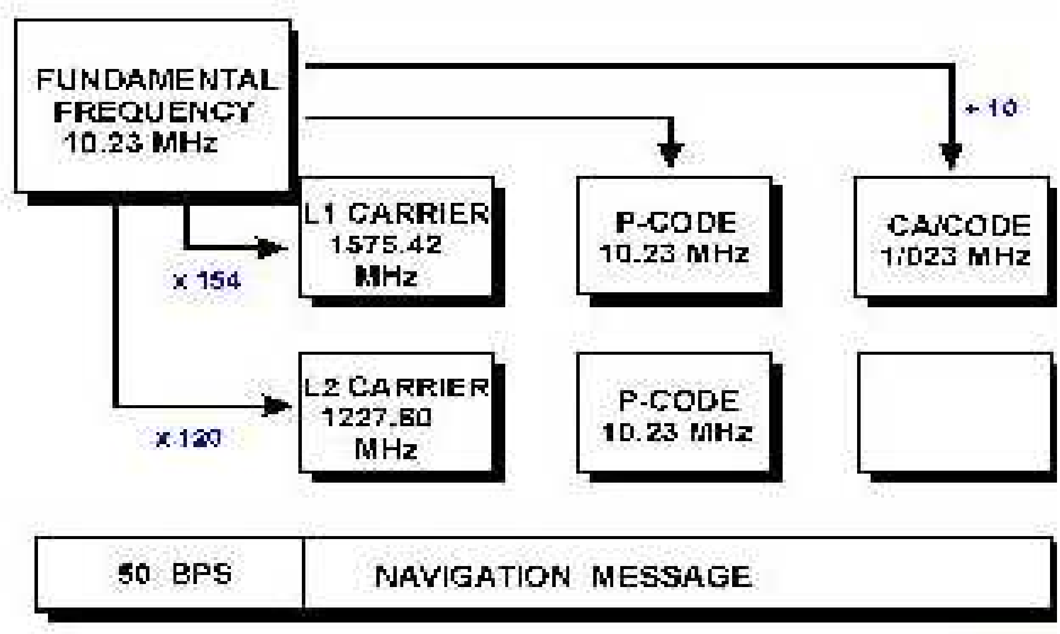


Figure 2.5: GPS satellite signal components.[1]

2.3.1 L-band Carrier Waves

As the name implies, the carrier waves provide the means by which the ranging codes and Navigation Message is transmitted to earth (and hence to the user). The primary function of the ranging codes is to permit the signal transit time (from satellite to receiver) to be

[1] From reference No. 8

determined. The Navigation Message is modulated on both carrier frequencies and contains the satellite ephemeris, satellite clock parameters, and other pertinent information such as general system status messages and an ionospheric delay model, necessary for real-time navigation to be performed. Each of these signal components are described below.

All signal components are derived from the output of a highly stable atomic clock (Figure 2.6 below). In the operational (Block II/IIR) GPS system each satellite is equipped with two cesium and two rubidium atomic clocks. (The Block IIF satellites may be equipped with a space-qualified hydrogen maser.) The clocks generate a pure sine wave at a frequency f0 = 10.23MHz, with a stability of the order of 1 part in 1013 over one day. This is referred to as the fundamental frequency.

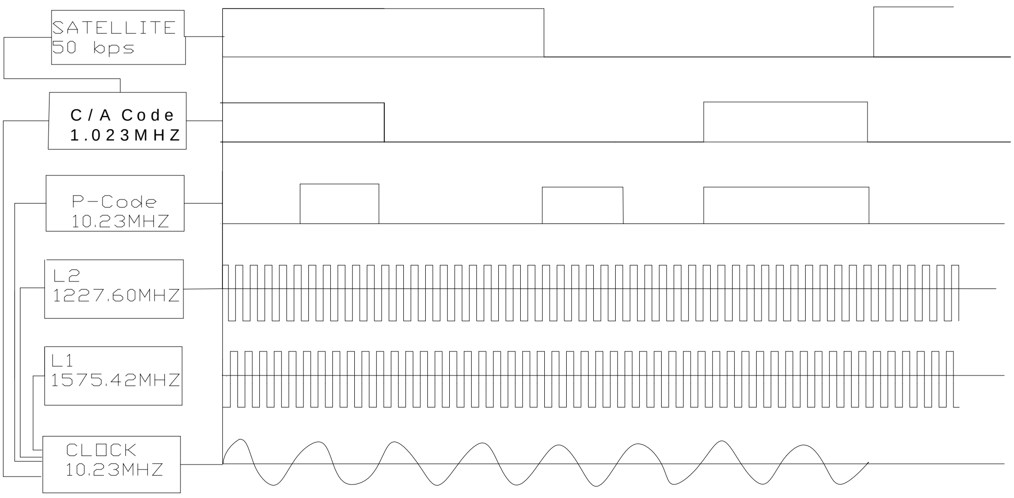


Figure 2.6: GPS signal component frequencies.[1]

Multiplying the fundamental frequency f0 by integer factors yields the two microwave L-band carrier waves L1 and L2 respectively (above two figures). The frequency of the two waves is obtained as follows:

$f_{L1} = f_0 \times 154 = 1575.42\text{MHz}$

$f_{L2} = f_0 \times 120 = 1227.60\text{MHz}$

[1] From reference No. 8

In the Global Positioning System there are two distinct codes used to modulate the L-band carriers, namely the ranging codes and the Navigation Message.

The L1 carrier was designed to be modulated with two codes, one intended for civilian use and the other reserved for the military, whereas the L2 carrier is modulated only with the military code. Both carriers also contain the Navigation Message.

2.3.2 PRN (pseudo-random-noise) Ranging Codes

Two ranging codes are used:

- The **C/A code**, the "clear/access" or "course/acquisition" code (sometimes also referred to as the "S code").
- The **P code**, the "privSate" or "precise" code, which under Anti-Spoofing (AS) is replaced by the "Y" code.

The C/A and P (or Y) codes can be considered as the measuring rods - they provide the means by which a GPS receiver can measure one-way distances to the satellites. Both codes have the characteristics of random noise, but are in fact binary codes generated by mathematical algorithms and are therefore referred to as "pseudo-random-noise" (or PRN) codes.

The C/A codes are 1023 "chip" long binary sequences, which are generated at a rate of 1.023 million chips per second, that is at a frequency of 1.023 MHz (see Figure 2.6). Hence the entire C/A code sequence repeats every millisecond. The "wavelength" of the code (length of the chip) is approximately 300 m, and the total sequence is therefore about 300km long. Each GPS satellite is assigned a unique C/A code.

The P code is a far more complex binary sequence, being approximately 266.4 days long with a chipping rate at the fundamental frequency f0 = 10.23 MHz. It is generated in an analogous manner to the C/A code. The "wavelength" of this code (length of the P code chip) is approximately 30m, ten times the resolution of the C/A code (Figure 2.7). Instead of assigning each satellite a unique code of its own, as is the case with the C/A code, the P code is allocated such that each satellite transmits a one week portion of the 266.4 day long sequence.

Under Anti-Spoofing the P code is encrypted through the modulation of a further secret code - the "W code".

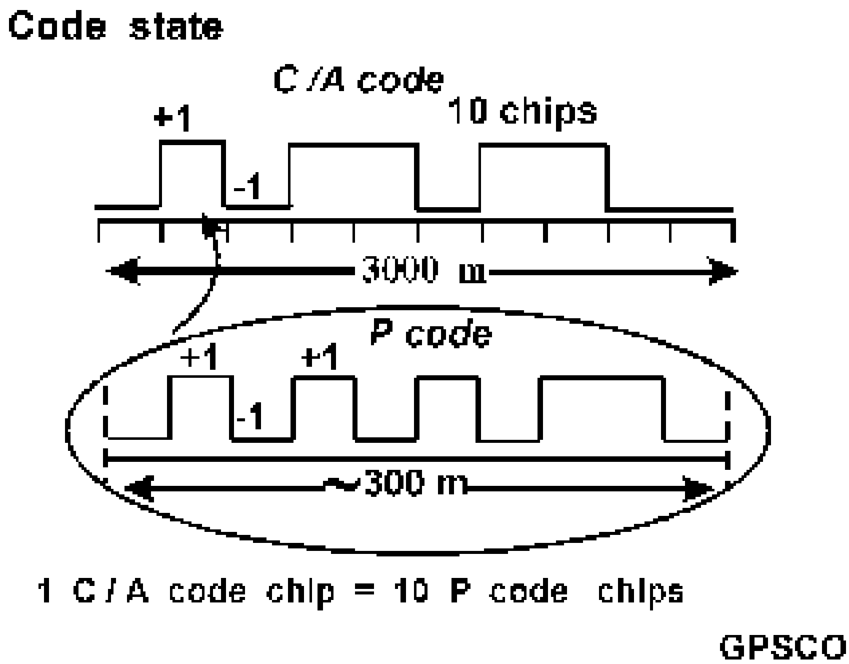


Figure 2.7: C/A and P code chip sequences.[1]

2.3.3 Navigation Message

In order for a GPS navigator to derive real-time position a Navigation Message is transmitted on both L-band frequencies.
The Control Segment uplinks this information into each satellite for subsequent transmission to all users on a regular (nominally daily) basis. The satellite message is in

[1] From reference No. 8

a binary form, like the ranging codes, but the sequence is *not* random. The message is transmitted at a rate of one bit ("0" or "1", as in a computer) see Table 2.2 .[1]

Table 2.2 NAVSTAR GPS Signal code and Carrier Frequencies(Block IIR)

			Code	
Carrier	Frequency (MHz)	Wavelength	Civilian C/A-Code	Military P(Y)-Code
L1	1575.42	19 cm.	Present 293m wavelength	Present 29.3m wavelength
L2	1227.60	24 cm	Not Present	Present 29.3m wavelength

Much like a radio station broadcast, several different types of information included in these signals like the satellite's identification, a GPS time stamp, and satellite position.

2.4 GPS Error Sources

It is important to understand the basic errors sources and their effects on the accuracy of the observations. Generally we can divide errors to three types:

2.4.1 Instrumental Errors:

Both the receivers and the satellites can contain errors:

1. Satellites clock errors uncorrected by Control Segment results errors.
2. GPS errors are a combination of noise, clock bias, and blunders.
   - Noise errors are the combined effect of PRN (Pseudo Random Noise) code and noise within the receiver noise.

[1] From reference No. 8

- Clock bias the difference between the clock's indicated time and true universal time.
- Blunders Errors, usually large, which arise from an avoidable error in observation or human error.

2.4.2 Natural Errors

1. Tropospheric delays: The troposphere is the lower part (ground level to from 8 to 13km) of the atmosphere that experiences the changes in temperature, pressure, and humidity associated with weather changes. Complex models of tropospheric delay require estimates or measurements of these parameters.
2. Ionosphere delays: The ionosphere is the layer of the atmosphere from 50 to 500 km that consists of ionized air.

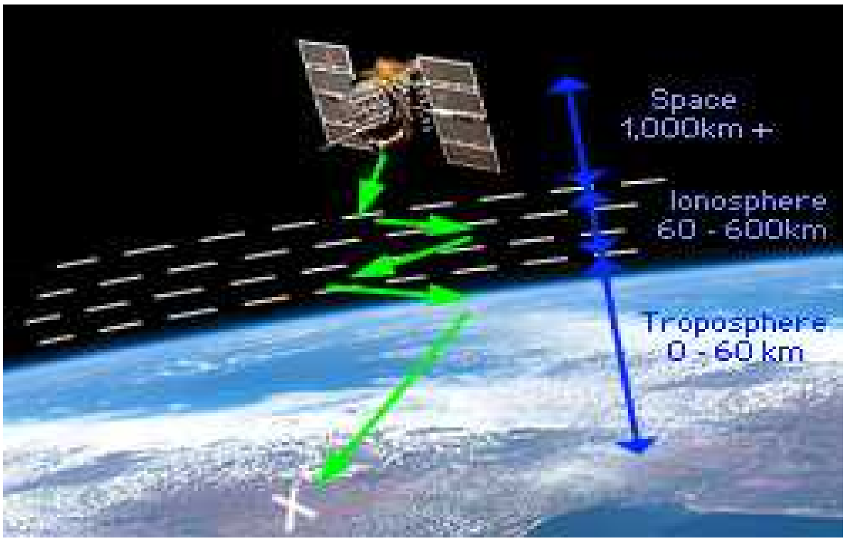


Figure 2.8: Tropospheric & Ionosphere error[1]

3. Multipath: is caused by reflected signals from surfaces near the receiver that can either interfere with or be mistaken for the signal that follows the straight-line path from the satellite. Multipath is difficult to detect and sometime hard to avoid.

[1] From reference No. 9

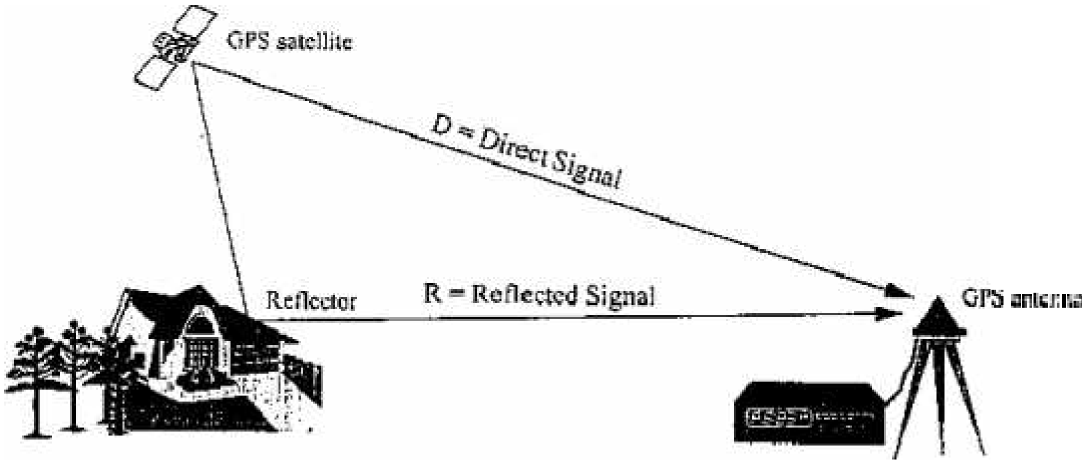


Figure 2.9: Multipath error [1]

2.4.3 Personal Errors

User mistakes, including incorrect geodetic datum selection, can cause errors from 1 to hundreds of meters range error budget is presented in Table 2-3 .The budget expressed for 95% probability level of the system.

Table 2.3: GPS System Range Error-Budget [2]

Error Source	Maximum Error
Ionosphere	10 Meters
Troposphere	1 Meters
Multipath Error	0.5 Meters
Satellite Clock	1 Meters
Measurement noise	0.5 Meters
Total	13 Meters

[1] From reference No. 8
[2] From reference No. 10

2.5 The Impact of Satellite Geometry

The accuracy with which positions can be determined is not just a function of the measurement precision. It is also a function of the satellites, receiver's geometry. Configuration geometry is usually expressed by the Dilution of Precision (DOP) factor. DOP is the ratio of the positioning accuracy to the measurement accuracy.

2.5.1 Dilution of Precision - DOP

- a function of the geometry of satellites
    1. satellites are clustered - poor geometry - poor triangulation
    2. satellites distributed across the sky - good geometry
- software on receiver can filter data with unacceptable DOPS
- numerous DOPs:

PDOP = Position Dilution of Precision (3-D).

$$\text{PDOP} = \sqrt{\pm X^2 + \pm Y^2 + \pm Z^2}$$

HDOP = Horizontal Dilution of Precision (Latitude, Longitude)

$$\text{HDOP} = \sqrt{\pm X^2 + \pm Y^2}$$

VDOP = Vertical Dilution of Precision (Height)

$$\text{VDOP} = \sqrt{\pm H^2}$$

TDOP = Time Dilution of Precision

$$\text{TDOP} = \sqrt{\pm T^2}$$

Where:

σX , σY , σZ : are the variances of the X, Y and Z components.

σH : are the variances of the height component.

σT : is the variance of the estimated receiver clock error parameter.

In the case of GPS point positioning, which requires the estimation of four parameters: 3-D position and receiver clock error, the most appropriate DOP factor is the Geometric Dilution of Precision (GDOP):

$$\text{GDOP} = \sqrt{(\text{P}DOP)^2 + (\text{T}DOP)^2}$$

GDOP can be interpreted as the reciprocal of the volume of a tetrahedron that is formed from the four satellites and receiver position; hence the best geometric situation for point positioning is when the volume is a maximum, which therefore requires GDOP to be a minimum. Figure 2-10 below illustrates the situation of good and poor GDOP.

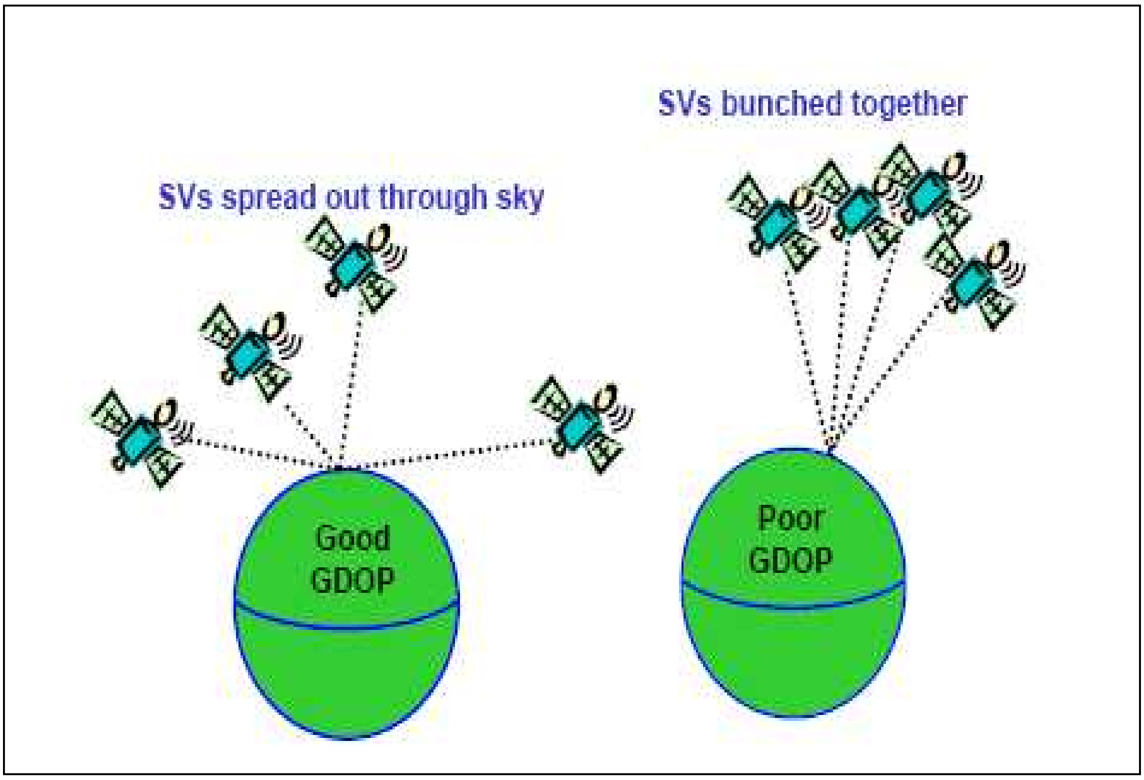
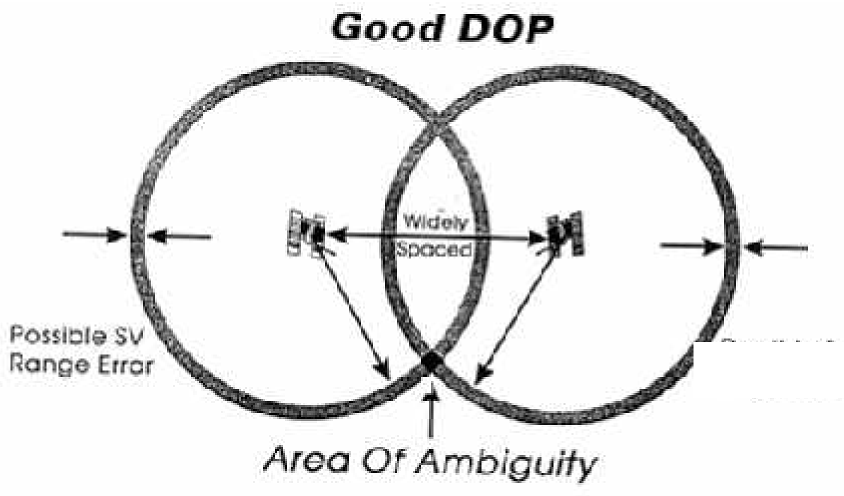


Figure 2.10: The Relationship between satellite configuration geometry and GDOP



Poor DOP

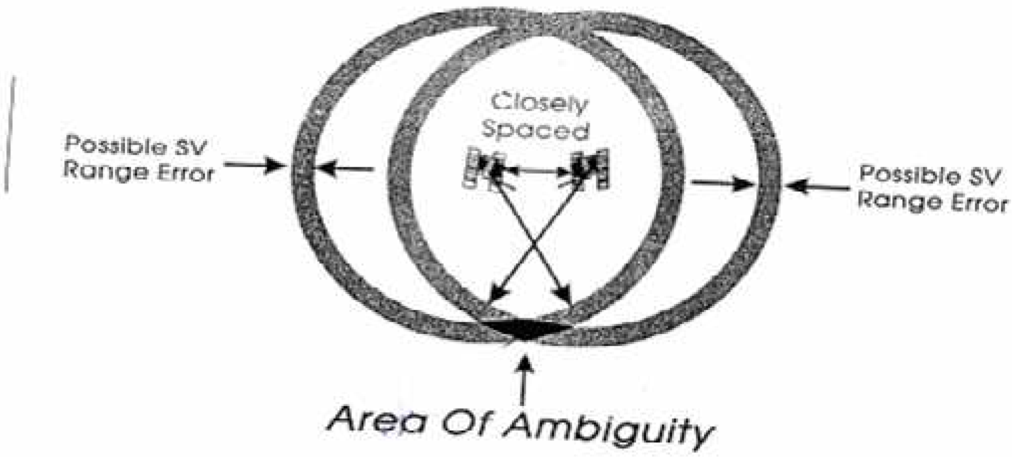


Figure 2.10: -Continue.

2.6 How GPS Works?

Satellite Trilateration

Coordinates are calculated for any position on earth by measuring the distances from a number of satellites to the position the satellites act as precise reference points. If the distance from one satellite is known, the position can be narrowed down to the surface of a sphere surrounding that satellite.

One measurement narrows down our position to the surface of a sphere (Figure 2.11)

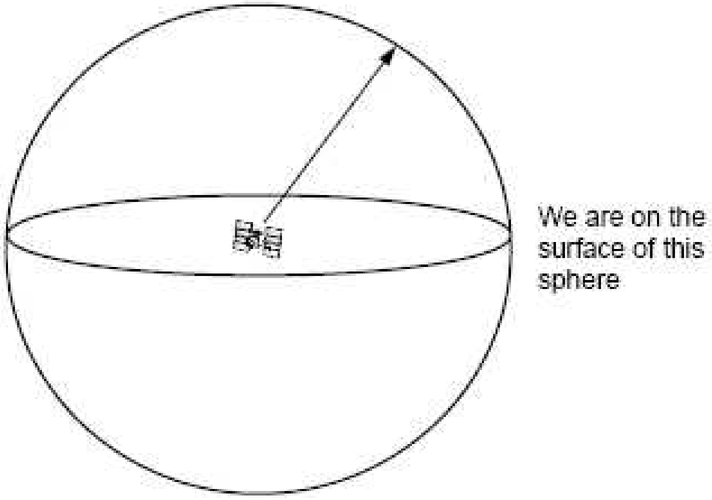


Figure 2.11: One satellite [1]

If the distance from a second satellite is also known, this narrows the position down to the intersection of the two spheres.

A second measurement narrows down our position (Figure 2.12)

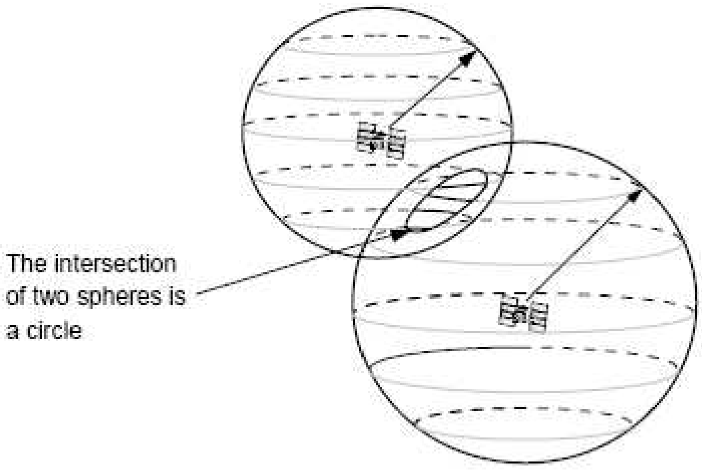


Figure 2.12: Two satellites [2]

[1] From reference No. 9

[2] From reference No. 9

Add a third satellite and the position is narrowed down to one of two points. A third measurement narrows our position down to two points (Figure 2.13)

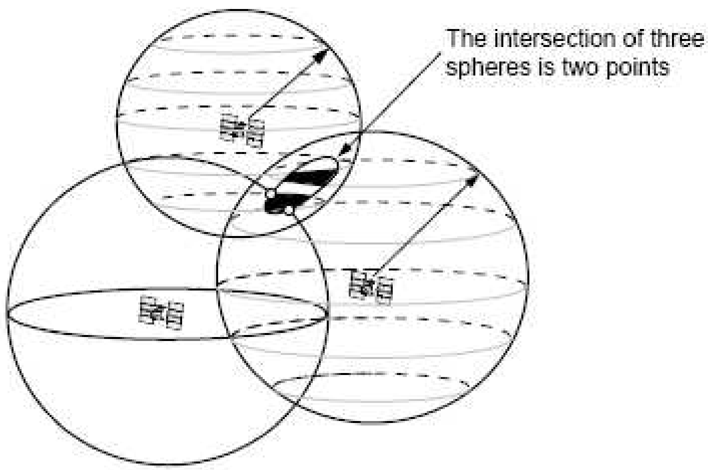


Figure 2.13: Three satellites [1]

One of these positions is disregarded it will be far out in space or moving at high speed and so by eliminating this position the correct answer can be found.
Although three satellites can be used to calculate the coordinates for a position, a fourth satellite is needed to solve the four unknowns, x, y, z, and time.

A fourth measurement narrows our position down to one point (Figure 2.14)

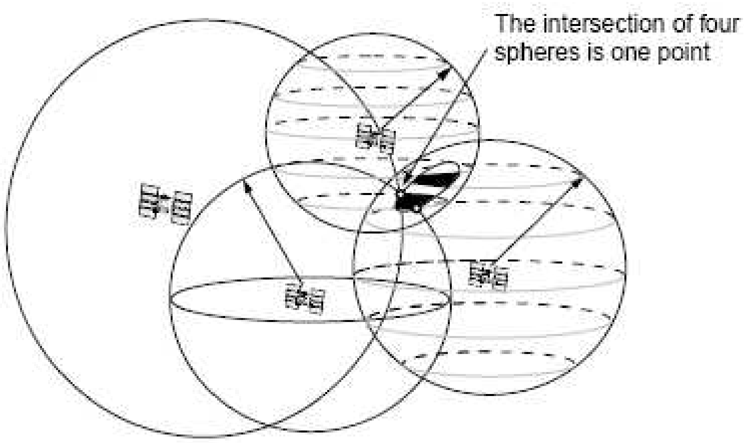


Figure 2.14: Four satellites [1]

[1] From reference No. 9

This explanation oversimplifies the problem, since we are really working with a three dimensional position. In mathematical terms, we have four unknowns: latitude longitude, altitude and time .In order to solve for four unknowns, we need four equations. This translates to a need for a minimum of four satellites for a unique solution, more than four satellites allow for more accurate positioning. With four satellites or more, we are not only able to locate ourselves, but we can calibrate our receiver's clock to the same accuracy as the atomic clocks on the satellites.

2.7 Other System (GALILEO and GLONASS)

Nowadays there are two world wide satellite systems - American GPS and Russian GLONASS see table 2.4 Comparing GPS and GLONASS. A new system - GALILEO will be constructed in Europe. The calculations were realized for the most possible GALILEO constellation 30 satellites distributed in 3 planes in an altitude 23222 km and with an inclination 54 degrees.

The planned development of the European GALILEO satellite navigation system alongside GPS and its various augmentation services extends the number of satellite position and timing services that will become available in a few years.

The constellation deployment phase will consist in gradually putting all the operational satellites into orbit from 2006 and in ensuring the full deployment of the ground infrastructure so as to be able to offer an operational service from 2008 onwards.

This could be particularly beneficial to critical and safety related applications such as civil aviation where total dependence upon a single navigation sensor poses considerable risk.

[1] From reference No. 9

Table2.4 Comparing GPS and GLONASS [1]

Parameter	GPS	GLONASS
satellites		
Number of satellites	24	24
Number of orbital planes	6	3
Satellites per orbital plane	4	8
Orbital inclination	55 degrees	64.8 degrees
Orbital radius	26560 Km	25510 Km
Orbital period	$11^{h}\,58^{m}$	$11^{h}\,15^{m}$
Signals		
Carrier	L1:1575.42 MHZ L2:1227.60 MHZ	L1:1602.5625—1615.5MHZ L2:1246.4375—1256.5MHZ
C/A-code(L1)	1.023 MHZ	0.511MHZ
P-code(L1,L2	10.23 MHZ	5.11 MHZ
General		
Time reference	UTC(USNO)	UTC(SU)
Geodetic datum	WGS 84	SGS 85

[1] From reference No. 3

CHAPTER THREE

GPS SATELLITE SURVEYING

This chapter includes GPS satellite surveying of Global Positioning System "GPS".

3-1 GPS Surveying Versus GPS Navigation

3.2 Some characteristics of GPS Satellite Surveying

3.3 Some characteristics of GPS Satellite Navigation

3.4 GPS Positioning Modes

3.5 Comments to the operational aspects of GPS Surveying

3.6 Factors Influencing the Adoption of GPS for Land Survey.

3.7 Observation Scheduling**.**

3.8 Instrumentation & Personnel Considerations.

3.9 Logistical Design Principles

3.10 Station Selection & Marking

3.11 Positioning Methods

3.12 Phase Differencing Techniques

CHAPTER THREE

GPS SATELLITE SURVEYING

3-1 GPS Surveying Versus GPS Navigation

The distinction between GPS Surveying and GPS Navigation can be made according to a variety of criteria, for example:

- According to "when", "where" and "how" the GPS technology is applied. This focuses on the applications, and the following simplistic distinction is therefore made: GPS Navigation supports the safe passage of a vessel or aircraft, from the port of departure, while underway and to its point of arrival; while GPS Surveying is mostly associated with the traditional functions of establishing geodetic control, supporting engineering construction, cadastral surveys and map making.
- According to operational aspects, such as the real-time, absolute positioning aspects of Navigation, as opposed to the post-processed, "unhurried", relative positioning characteristics of GPS Surveying.
- According to the type of measurement made and the GPS instrumentation used. GPS Navigation-type receivers are comparatively low-cost, code-correlating instruments that only measure pseudo-range, whereas GPS Surveying receivers are expensive, phase measuring instruments that include many special features and complex software in order to support their function.
- According to the mathematical models used. For example, because the primary measurement in GPS Navigation is the pseudo-range, the biases are dealt with in a more "casual" fashion (with the exception of the clock errors, they are all ignored!). In contrast, GPS Surveying requires a more careful treatment of the biases during the data processing.

3.2 Some characteristics of GPS Satellite Surveying are

- The points being coordinated are stationary.
- GPS data are collected over some "observation session".
- Relative positioning modes of operation, and hence high accuracies.
- The measurements are made on the L-band carrier wave, hence requiring special instrumentation and software.
- Mostly associated with the traditional surveying and mapping functions.

3.3 Some characteristics of GPS Satellite Navigation are

- The points being coordinated are generally in motion.
- GPS is collected for an "instant", and the solution is obtained in real-time.
- Absolute and relative positioning modes of operation, of comparatively low accuracy.
- The measurements are typically made on the PRN codes, and require the processing of pseudo-range data.
- Mostly associated with defining safe passage of ships and aircraft.

3.4 GPS Positioning Modes

- STATIC positioning: coordination of stationary points, either in absolute or relative mode. This is generally synonymous with the SURVEYING mode of positioning, based on the analysis of carrier phase observations.
- KINEMATIC positioning: coordination of moving points, either in absolute or relative mode. This is generally the NAVIGATION mode of positioning, based on pseudo-range observations.
- RTK positioning: Real Time Kinematic GPS has been adopted across a broad range of engineering survey activities. From its incept as a tool to densify site control RTK has moved quickly to become the preferred system for

engineering stake-out and measure and is now making rapid in-roads in the new and innovative application on construction and earthmoving equipment

3.4.1 Static surveying

Static used for long lines, geodetic network, and tectonic plate studies etc, offers high accuracy over long distances but is comparatively slow.
This was the first method to be developed for GPS surveying. It can be used for long baselines (usually 20 km an over) in this method two or more GPS receivers at two or more stations simultaneously receive signals from a minimum of four satellites. One receiver is placed on point whose coordinate are known accurately in UTM or local.

The other receivers is placed on the end of the other end of the baselines and is known as the rover, data is then recorded at both stations simultaneously it is important that data is being recorded at the same rate epoch at each station.

The data collection rate may be typically set to 15, 30 or 60 seconds, occupation time depends on the type of GPS receivers, the separation distances between receivers, the ionospheric activity, number of satellites and the geometry .

As a rule of thumb, the observation time is a minimum of 1hour for 20 km line with 5 satellites longer line required, observation time. Good coordination is required between the survey crews in order to maximize the potential of having three receivers.

3.4.2 kinematic surveying

Used for detail survey and measuring many point in quick succession. Very efficient way measuring many point are close together. However if there are obstructions to the sky such as bridges, trees, tall building etc, and less than 4 satellite are tracked, the equipment must be reinitialized which can take 5-10 minutes.

The kinematic technique is typically used for detail surveying recording. The technique involves a moving rover. Position can be calculated relative to the reference (base).

The kinematic surveying is that receivers must maintain a lock to minimum of four satellite the reference and rover are switched on and remain absolutely stationary for (5-20) minutes, collecting data, the actual time depends on the baseline from the reference and the number of satellite observed after this period the rover may then move freely.

A major point to watch during kinematic surveys is to avoid moving too close to objects that could block the satellite signal from the rover receiver. If any time less than four satellite are tracked by the rover receiver ,you must stop, move into a position where four or more satellite are tracked and perform an initialization again before continuing .

3.4.3 (RTK) real-time kinematic GPS

Uses a radio data link to transmit satellite data from the reference to the rover this enables coordinates to be calculated and displayed in real time, as the survey is being carried out used for similar application as kinematic. A very effective way for measuring detail as results are presented as work is carried out. This technique is however reliant upon a radio link, which is subject to interference from other sources and also line of sight blockage.

The reference station has a radio link attached a rebroadcasts the data it receives from the satellite. The rover also has a radio link and receives the signal broadcast from the reference. The rover also receives satellite data directly from the satellite via its own GPS Antenna. These two sets of data can be processed together at the rover to resolve the ambiguity and therefore obtain a very accurate position relative to the Reference receiver.

Once the Reference Receiver has been set up and is broadcasting data through the radio link, the Rover Receiver can be activated. When it is tracking satellites and receiving data from the Reference, it can begin the initialization process.

Once the initialization is complete, the ambiguities are resolved and the Rover can record point and coordinate data. At this time, baseline accuracies will be in the (1-5 cm) range. It is important to maintain contact with the Reference Receiver; otherwise the rover may lose the ambiguity. This results in a far less accurate position being calculated. Additionally, problems may be encountered when surveying close to obstructions such as tall buildings, trees, etc. as the satellite signal may be blocked.

RTK is quickly becoming the most common method of carrying out high precision high accuracy GPS surveys in small areas and can be used for similar applications, as a conventional total station.

3.5 Comments to the Operational Aspects of GPS Surveying

3.5.1 Survey planning considerations are derived from

- The nature and aim of the survey project.
- The unique characteristics of GPS, and in particular no requirement for station indivisibility.
- The number of points to be surveyed, the resources at the surveyor's disposal, and the strategy to be used for propagating the survey.
- Prudent survey practice, requiring redundancy and check measurements to be incorporated into the network design.

3.5.2 Field operations are characterized by requirements for

- Setup of antennas over predefined ground marks.
- Simultaneous operation of two or more GPS receivers.

- Coordinate data gathering operation so that data collected has the same time-tags, involves the same satellites, etc.
- Common data collection over some observation session.
- Coordinated demount of GPS antennas and transport to new stations.

3.5.3 Field validation of data collected, in order to

- Verify sufficient common data collected all sites operating simultaneously.
- Verify quality of data to ensure that acceptable results will be obtained.
- Where data dropout is high or a station has not collected sufficient data, reoccupation may be necessary.

3.5.4 Office calculations

To obtain GPS solutions for single sessions or baselines.

- To combine the results of single sessions into a network solution.
- To incorporate external information (for example, local control station coordinates), and hence modify the GPS-only network solution.
- To transform the GPS results to the local geodetic datum, and to derive orthometric heights.
- To verify the accuracy and reliability of the GPS survey.

The quality of the baseline vector solution is dependent on, amongst other things

1. Length of baseline.
2. Single or dual-frequency instrumentation.
3. Length of observation session.
4. Number of observed satellites.
5. Observable being processed.
6. Processing software.
7. Quality of ancillary information (orbits, fixed sites, etc.).

Typical horizontal baseline accuracy is expressed as:

$$\mathbf{e = a + (b\,.L)}\ [1] \quad \text{---} \quad \mathbf{(3.1)}$$

Where: e relative error

L baseline length in km..

a = 0.2 - 1 cm (centring error).

b = 1 - 5 ppm.

3.6 Factors Influencing The Adoption Of GPS For Land Survey.

GPS does not need to be only competitive against conventional terrestrial techniques of surveying, but also other extraterrestrial techniques and new technologies. GPS relative positioning technology can, in principle, be employed for a wide range of activities. Applications are classified into three general categories.

1-Class A (Scientific):	better than 1 ppm
2-Class B (Geodetic):	1 to 10 ppm
3-Class C (General Surveying):	Lower than 10 ppm.

3.6.1 GPS Versus Conventional Terrestrial Surveying ADVANTAGES

- Operations are weather independent
- Network independent site selection, hence sites placed where needed
- Around-the-clock operation
- Economic advantages from greater efficiency and speed of survey
- Geodetic accuracies easily achieved
- 3-D coordinates are obtained

[1] From Reference No. 8.

3.6.2 GPS Versus Conventional Terrestrial Surveying Disadvantages

- High productivity places greater demand on survey planning and logistical considerations.
- No sky obstructions can be tolerated therefore cannot be used underground, under foliage or structures.
- GPS surveying is generally "targeted" to satisfy a specific survey need.
- No azimuth control for subsequent non-GPS surveys.
- Horizontal and vertical coordinates from GPS must be transformed if they are to be useful for conventional survey applications.
- GPS accuracies are generally higher than the surrounding existing control.
- High capital cost of GPS instrumentation.
- New skills needed

3.6.3 Accuracy of Known Stations

Both GPS derived coordinates and the coordinates of geodetic control are essentially RELATIVE. However, GPS derived coordinate accuracy relative to the local geodetic datum origin depends on:

- The datum station.
- The transformation process.
- Any secondary distortion of the GPS network resulting from fitting to the datum defined locally by the known geodetic stations.

3.6.4 Network Shape

- As in the case of conventional surveys, there is an impact arising from "structural" considerations.
- Some networks are superior to others with regard to "strength".

- Only independent baselines contribute to network strength.

GPS networks may have different shapes (Figure 3-1), as well as different "strengths" arising from the number of independent baselines observed over a number of sessions

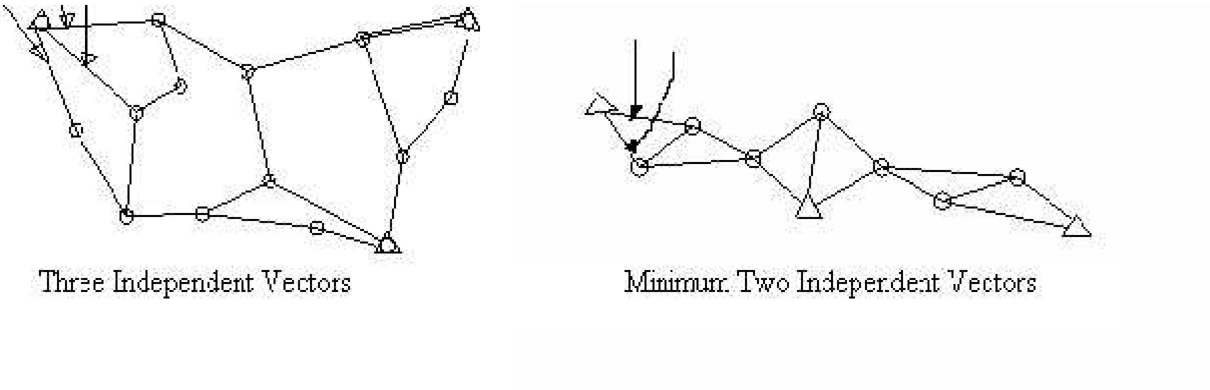


Figure 3.1: Network shapes "wide and narrow" [1].

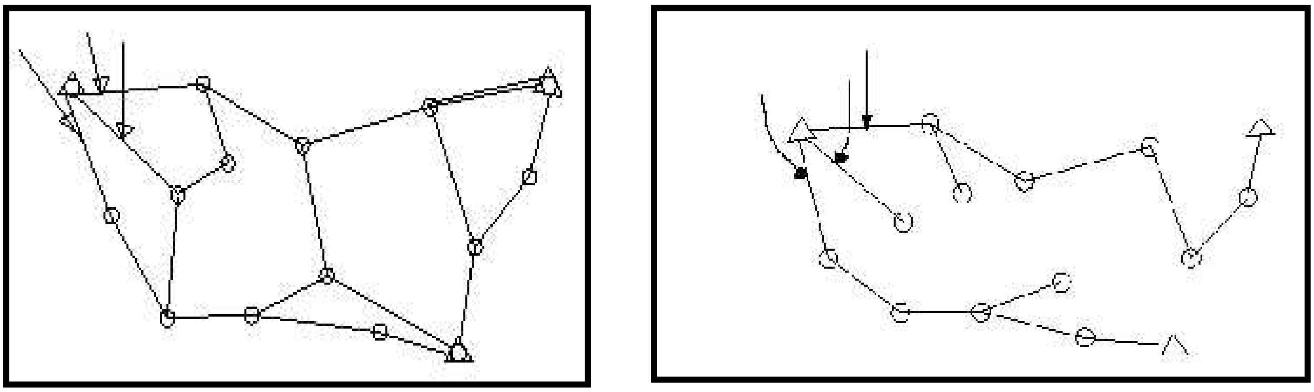


Figure 3.2: Network strength [2]

A function of the number and location of independent baselines.

Overall network quality

- Baseline quality.
- Homogeneity of GPS survey (single and dual-frequency results, etc.).
- Number and distribution of independent baselines.

---

[1] From Reference No. 8.
[2] From Reference No. 8.

- Other geodetic observations (distance, angle, etc).

The overall network accuracy is a function of the number and distribution of repeat baselines and redundant station occupations.

3.7 Observation Scheduling.

There are three considerations:

- Those that relate to the satellites themselves: how many to observe, for how long, etc.
- Those that relate to satellite-receiver geometry.
- Those that relate to logistical design: number of observation sessions per day, number of multiple site occupancies, etc.

3.7.1 Satellite Considerations.

To prepare an observation schedule for a GPS survey it is necessary to first define the satellite constellation to be tracked, including such information as:

- Rise and set times of satellites above the observing horizon of a site. (Due to atmospheric refraction modelling difficulties GPS satellites are not normally tracked at elevations less than 15° to 20° above the horizon.)
- Health of the satellites. Each satellite broadcasts a health status indicator within its Navigation Message. This should be monitored during tracking, but a history of health problems may indicate a satellite which should be avoided if at all possible. (Note that although a satellite may be "unhealthy" for GPS navigation this does not mean that it cannot be used for surveying.)
- Satellite-site geometry: azimuth and elevation of satellites as a function of time.
- Information on the status of satellites, for example, planned orbit man oeuvres, shutdowns, testing, new satellite launches, etc.

3.7.2 Measures of Satellite Geometry

The accuracy of GPS-derived coordinates is generally a function of:

1. The measurement precision,
2. The systematic errors present,
3. The processing strategy used, and
4. The receiver-satellite geometry during the observing session.

Of these, items (2) and (4) are the most variable. The receiver-satellite geometry is highly predictable and is manifest in the Normal Equations for the Least Squares solution for the site coordinates. In fact the elements of the design matrix (the unit vectors for the receiver-satellite in question) can be computed beforehand and, similar to the planning procedures used for conventional terrestrial surveys. Hence, given a satellite constellation, the approximate coordinates of the receivers, the time of day and length of session, the Normal Equation System can be determined, inverted and the formal errors of the estimable parameters obtained.

3.7.3 Some Logistical Factors.

Observation scheduling within a network relates to:

- The number of observation sessions in a day (dependent on length of the work day, and minimum session length).
- Total non-productive times: travel time between stations, data downloading
- Number of occupations of each station (new and unknown).

3.8 Instrumentation & Personnel Considerations.

Instrumentation considerations as they relate to the project planning process include:

- Number of available GPS receivers: the larger the number of receivers in a session, the larger the number of directly connected stations, and hence a better network, faster progress and a less expensive survey. However, there may be a restriction on the availability of receivers and field parties, and, in addition, the logistical problems quickly multiply. The optimum number of receivers appears to be of the order of four to six.

- Receiver type: all geodetic GPS receivers produce, in principle, similar datasets and hence similar final accuracies.
- Single or dual-frequency receivers: dual-frequency instruments permit compensation for the ionospheric delays on the GPS signals; hence they are essential for high accuracy applications. They are usually of little benefit for baselines <30km. dual-frequency instrumentation is generally necessary if modern "rapid static" survey techniques are being used.

3.9 Logistical Design Principles

With experience, the organizational design of a session-by-session observation schedule is a straightforward matter based on a few simple "rules-of-thumb" Receiver deployment schemes.

The "shape" of a GPS session, or final network (multi-session), plays little part in the final accuracy, unlike the situation with conventional surveys. Length of lines in a session do influence the final accuracy, both through the "ppm" relationship, as well as a change in the "ppm" value at distances above which ambiguity resolution can be carried out.

Adjacent stations should be connected directly that is, keep baselines short. Stations on the perimeter of project area should be directly connected. Connect all stations in network through "link" sites common to two or more sessions, so that a minimally constrained GPS network is established. The percentage of multiple occupancies is directly related to the accuracy classification of the survey, and is usually defined in the prescribed "standards & specifications"

Maximize geometrical redundancy through having multiple occupancy of sites (though only within reason). There are two types of redundancy Reoccupation of several sites at the same time to define repeat baselines (to allow for checks on the internal consistency of GPS surveys). Form loops of stations occupied during different sessions, permitting checks on loop closure statistics.

Where possible sites should be revisited by different field parties, hence ensuring independent setups, in order to minimize the chance of misidentifying the station mark. Avoid no check baselines (Figure 3 below), where one station of the baseline has only been visited once.

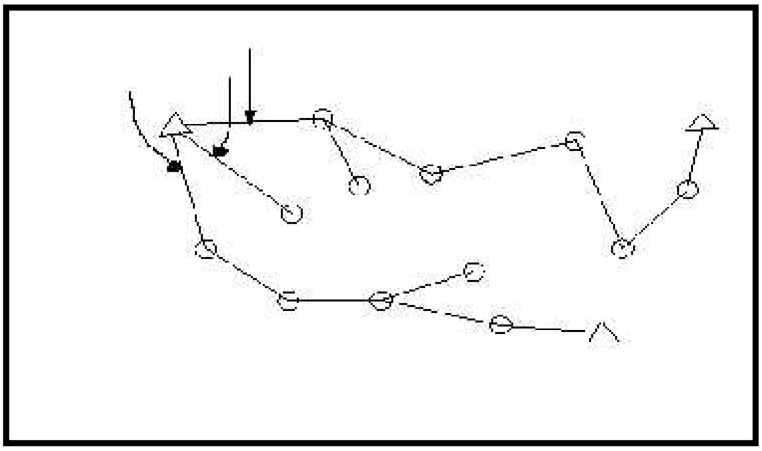


Figure 3.3: Factoring in redundancy to prevent "no check" baselines. [1]

3.10 Station Selection & Marking

This information should be clearly stated in words as well as described in some graphical form. This is critical for minimizing down-time due to difficulties in finding stations, or if access involves caretakers (for example, to visit the roofs of buildings), etc.

As with details of the station access and point description, the area around the site should be studied carefully. Depending on the aim of the project, and its accuracy requirements, this task may be very elaborate and include, for example:

- Investigating the provision of on-site power.

[1] From Reference No. 8.

- Testing soil stability and defining the appropriate antenna mount (tripod, pillar, etc.).
- Noting the presence of any potential multipath causing structures.
- Noting any UHF, TV, radio, microwave or radar transmitters (they could affect a receiver's operation).
- Establishing permanent monumentation using previous marks helps avoid this.
- Establishing nearby azimuth marks.
- Clearing the area of possible obstructions caused by trees or shrubbery.
- Taking photographs of the surrounding area, including any tree cover.

What is a good site?

- No signal obstructions above 20°.
- No multipath causing surfaces, such as metallic fences, structures and water surfaces.
- No nearby electrical installations, such as high tension cables, radio/radar/TV transmitters**.**

3.11 Positioning Methods

The positional accuracy that the user obtains depends on the GPS frequencies or code utilized (L1and L2 carriers, C/A, and P code) and the surveying technique employed (point positioning or relative positioning).

Method 1: Point positioning

One receiver is used in point positioning to determine the three dimensional position of an unknown location by making measurement of the (C/A or P) code phases. Observational model for point positioning. The unknown receiver location(X,Y,Z or Φ ,λ ,h) is computed by distance resection using pseudo ranges from a minimum of four satellite (Figure 3-4). A minimum of four satellites is required to compute the unknown three dimensional receiver's coordinates of the antenna center and receiver clock error (XA, YA, ZA, δðt $_A$). The pseudo range observable is defined as

$P^i_A = \rho^i_A + C(\delta t^i - \delta t_A) + \delta^i_A iono + \delta^i_A trop + \Sigma$ --------------------------------------(3.2)

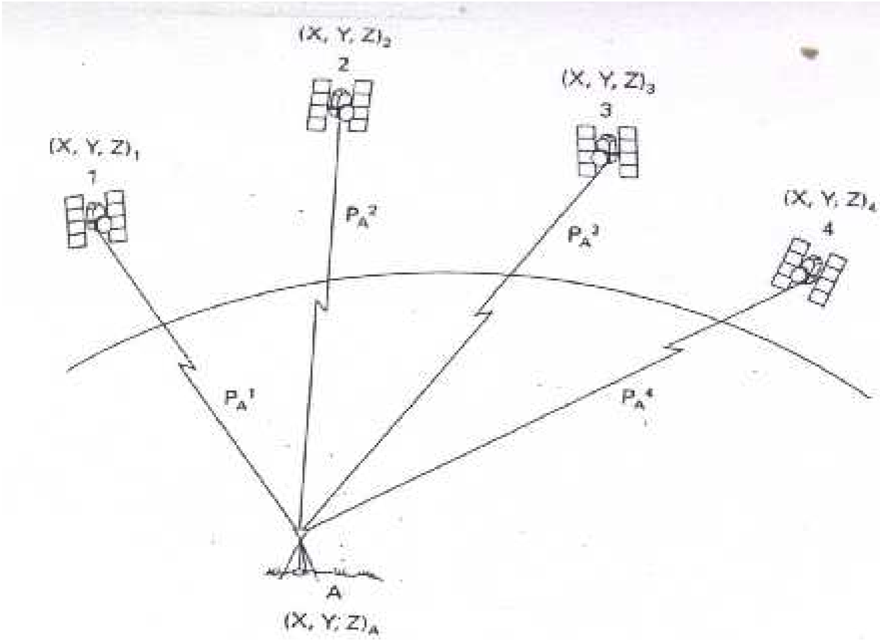


Figure 3.4: Four pseudo ranges from four satellites.[1]

In which the superscripts designate the satellite and the subscripts denote the receiver

$P^i_A = C(t_A - t^i)$ = measured pseudo range from satellite i to receiver A (m)

t_A = time of reception of signal (sec). δt^i= satellite-clock error (sec).

t^i= time of transmitting signal (sec). δt_A = receiver-clock error (sec).

c= vacuum speed of light (m/s). δ^i_A iono= ionospheric error (m).

δ^i_A trop = tropospheic error (m). Σ = receiver noise (m).

ρ^i_A = geometric rang (m) = $[(X^i - X_A)^2 + (Y^i - Y_A)^2 + (Z^i - Z_A)^2]^{4/2}$ in which

[1] From Reference No. 4.

X^i , Y^i , Z^i = satellite location and X_A , Y_A , Z_A = receiver location.

The ionospheric error can be reduced using correction transmitted within sub frame 1.Also given a dual frequency receiver, frequencies L1and L2 can be used to compensate for the ionospheric error. The effects of ionospheric and tropospheic delays can be reduced by not observing a low elevation angles.

A correction for the tropospheic error is possible using mathematical model. The satellite clock error can be corrected using parameter form subframe 1of the data message. After making this correction, the equation for range is

$$P^i_A = [(X^i - X_A)^2 + (Y^i - Y_A)^2 + (Z^i - Z_A)^2]^{4/2} - c\,\delta t_A \text{ ---------------------------(3.3)}$$

When the ranges from satellite are observed simultaneously, for equation (3.3) are formed and the unknown receiver location (XA, YA, ZA) and clock error, δt_A , for that epoch can be computed. If ranges are measured to more four satellites, a least-squares adjustment is possible.

Method 2: Relative positioning by carrier phase measurements

Using carrier phase measurements is that the signal is ambiguous. The receiver accurately measures the phase difference between the incoming signal from the satellite and similar signal generated by the oscillator within the receiver. If ambiguity is resolved correctly, relative positioning using carrier phases provides the user with millimeter to centimeter results.

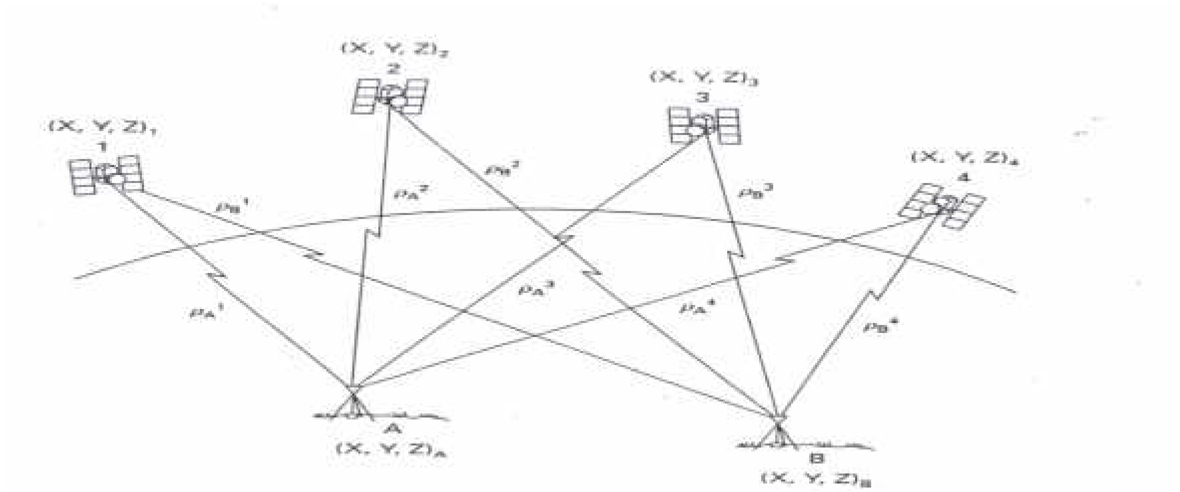


Figure 3.5: Eight pseudo ranges from four satellites to two stations for relative positioning

Observational model for carrier phase positioning the equation for carrier phase measurement used in relative positioning and scaled into cycles is

$$\Phi^{i}{}_{A} = f\delta^{i} - (1/\lambda)\,(P^{i}{}_{A}) - f\,\delta_{A} + N^{i}{}_{A} - f\delta^{i}{}_{A}\,\text{iono} + f\delta^{i}{}_{A}\,\text{trop} + \Sigma \text{-------------} \quad (3.4)$$

In which

$\Phi^{i}{}_{A}$ = carrier phase measurement, in cycles, for satellite i and receiver A

$P^{i}{}_{A}$ =geometric range in m. c= vacuum speed of light (m/s).

δ^{i} = satellite-clock error (sec). δ_{A} = receiver-clock error (sec).

$\delta^{i}{}_{A}$ iono= ionospheric correction (cycles). $\delta^{i}{}_{A}$ trop = tropospheic delay (cycles).

λ = carrier wavelength, in m (L1orL2). $N^{i}{}_{A}$ = integer ambiguity (cycles).

f = the fundamental satellite frequency (HZ). Σ = receiver noise (cycles).

3.12 Phase Differencing Techniques

The carrier phase measurement is very precise and is used for sub centimetric GPS positioning. Many of the errors associated with GPS must be eliminated, reduced, or modelled within software. In order to do this and to compute the ambiguities a number of processing strategies have been adopted.

3.12.1 Single difference

Consider two receiver (A and B)simultaneously tracking satellite i (Fig 3.6).The single difference is formed by subtracting the carrier phase equation (3.4) for the range between receiver A and satellite i from the carrier phase equation (3.4) for the range between receiver B and satellite i. This single difference equation is

$$\Delta\Phi^{i}_{AB} = \Phi^{i}_{B} - \Phi^{i}_{A}$$

$$\Delta\Phi^{i}_{AB} = [f\,\delta^{i} - (\rho^{i}_{B}\ /a) - f\,\delta_{B} + N^{i}_{B}] - [f\,\delta^{i} - (\rho^{i}_{A}\ /a) - f\,\delta_{A} + N^{i}_{A}]$$

$$\Delta\Phi^{i}_{AB} = (\rho^{i}_{A} - \rho^{i}_{B})\big/a \quad - f\,(\delta_{B} - \delta_{A}) + \Delta\,N^{i}_{A\,B} \qquad \text{---------------} \ (3.5)$$

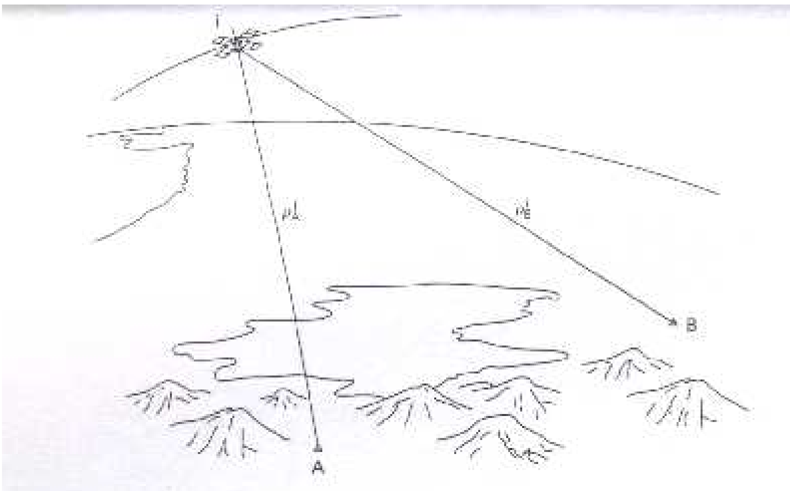


Figure 3.6: Single difference, satellite clock offset eliminated.[1]

3.12.2 Double difference

The double difference consists of taking difference between satellite and receiver (figure 3.7).The double difference operator is defined by the symbol
Where the superscript j denoted the reference satellite. The double difference equation as formed

$$\nabla\Delta \quad {}_{AB}{}^{i}\,\Phi - \ \Delta_{AB}{}^{j}\ \Phi\ \Delta\ = \ {}_{AB}{}^{ij}\ \Phi$$

$$\nabla\Delta\ \Big]\ {}_{B\ A}{}^{j}\ ) + \Delta\,N\,{}_{A} - \delta\,{}_{B}\ \Big) - f\,(\delta\ \big/a\,)_{A}\ \ {}^{j}\ - \rho\,{}_{B}{}^{j}\ = -\Big[\Big(\ (\rho\quad {}_{AB}{}^{ij}\ \Phi$$

---

[1] From Reference No. 4.

$$]_{B\ A}{}^{i}\ ) + \Delta\ N_{A} - \delta_{B}\ \Big) - f\ (\delta \diagup a\ )_{A}{}^{i}\ - \rho_{B}{}^{i} \qquad = -\Big[-\Big(\ (\ \rho$$

$$\nabla\Delta + \diagup a\ \Big)_{A}{}^{i}\ + \rho_{B}{}^{i}\ \rho^{i}\ -_{B}{}^{j}\ - \rho_{B}{}^{j} = -\Big[(\ \rho_{AB}{}^{ij}\ \Phi\nabla\Delta \quad \Big]_{AB}{}^{ij}\ N$$

$)_{B}$, δ_{A} the double difference eliminates the receiver clock error (δ

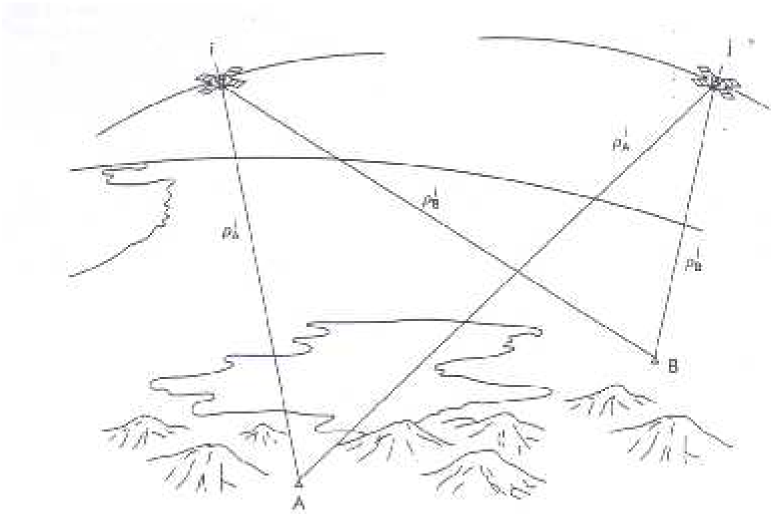


Figure 3.7: Double difference, receiver and clock offsets cancel. [1]

3.12.3 Triple difference

The condition for triple difference equation are shown in the figure(3. 8) where and receiver at stations A and B receiver signals first from satellite (I1) and(J1) then from the same satellite at (I2)and(J2).

Subtraction of the double difference Equation (3.6)for the first epoch ($\nabla\Delta$ 1)$_{AB}{}^{ij}$ Φ

from the second epoch ($_{AB}{}^{ij}$ $\Phi\nabla\Delta$ 2) yields the triple equation defined by the symbol $\delta\nabla\Delta\Phi^{ij}{}_{AB}$

$$\delta\nabla\Delta\Phi^{ij}{}_{AB} = -\Big[\ (\rho^{j2}{}_{B} - \rho^{j2}{}_{A} - \rho^{i2}{}_{B} + \rho^{i2}{}_{A}\ ) \diagup a \quad + \nabla\Delta N^{ij}{}_{AB}\Big]$$

---

[1] From Reference No. 4.

$$- \left[-(\rho^{j1}_{B} - \rho^{j1}_{A} - \rho^{i1}_{B} + \rho^{i1}_{A}) \big/ a \quad + \nabla\Delta N^{ij}_{AB} \right]$$

$$\delta\nabla\Delta\Phi^{ij}_{AB} = - \left[(\rho^{j2}_{B} - \rho^{j2}_{A} - \rho^{i2}_{B} + \rho^{i2}_{A} - \rho^{j1}_{B} + \rho^{j1}_{A} + \rho^{i1}_{B} - \rho^{i1}_{A}) \big/ a \right] \text{------ (3.7)}$$

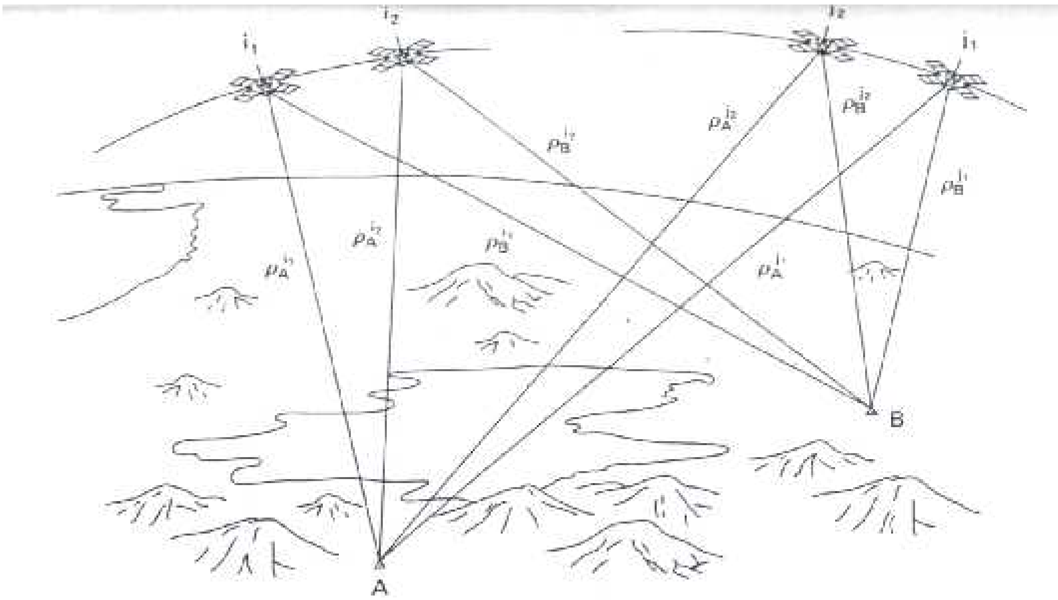


Figure 3.8: Triple difference, phase ambiguity eliminated [1]

[1] From Reference No. 8.

CHAPTER FOUR

SOFTWARE AND 5700 GPS RECEIVER

This chapter includes software and 5700 GPS receiver.

4.1 Trimble Geomatics Office v1.6 Software.

4.2 5700 GPS Receiver.

4.3 Setup Receiver and Start Static& RTK Survey.

CHAPTER FOUR

SOFTWARE AND 5700 GPS RECEIVER

4.1 Trimble Geomatics Office v1.6 Software

The Trimble Geomatics Office software is a key component in the Trimble Toolbox of Integrated Surveying solutions. Never has the collection, processing, and management of survey data been so easy. Surveyors and engineers who work with data received from GPS, digital levels, laser instruments, road design packages or GIS databases, appreciate its ease of use and common interface for all operations.

Trimble Geomatics Office software provides a seamless link from design to field to completion, with powerful processing capabilities to generate the final data in whatever form is desired.

4.1.1 Complete Survey Data Processing

The Trimble Geomatics Office software takes land survey office software one step further by integrating common tasks into a single, unified package tasks such as:

- Processing of GPS, conventional, and digital level survey data.
- Quality assurance and quality control of data (QA/QC).
- Road design data import and export.
- Survey data import and export.
- Digital terrain modeling and contouring.
- Datum transformation and projections.
- GIS data capture and data export.
- Selecting points and observations.
- Feature coding.
- Project reporting.

- Survey project management.
- GPS baseline processing.
- Survey network adjustment for GPS and conventional data.

With numerous innovative and unique features, the Trimble Geomatics Office software is exceptionally easy to use, intuitive and flexible. Visualization tools such as background maps and the Survey Plan views help you see the data in the context of the whole project. And powerful spatial data management capabilities bring a new level of productivity and efficiency to the surveying and civil engineering office.

4.1.2 Intuitive Visual Interface

In addition to the standard and familiar Windows interface, the Trimble Geomatics Office software features a Project Bar and Tool Bar, both set out to follow your workflow and designed to make the software easy to learn and use. Full Microsoft IntelliMouse support literally puts real-time zoom and pan functions at your fingertips.
The survey view and plan view allow you to switch seamlessly between a view of the survey displayed as observations and a view displayed as features on a plan. The survey view shows you survey information, such as RTK base stations, conventional setups and control points. The survey view shows you survey information, such as RTK base stations, conventional setups and control points.
After processing feature codes, the plan view shows topographic information, such as trees, fence lines, and roads. This complementary representation of data enables office operators to visualize what happened in the field, even if they did not collect the field data. The survey view is used to view raw data, analyze survey observations, process GPS baselines, and perform leasts-quares network adjustments. GPS observations are shown as vectors and optical observations are shown as observations from the instrument point.
Different observation types, such as post processed GPS, real-time GPS, conventional, laser, etc., are color-coded for instant recognition. View filters allow observation types to be displayed or hidden in order to concentrate on a specific set of observations.

In the plan view, the survey is displayed as a plot with features, point styles, lines and text annotations just as it would be in a final plan. You can use this view to process feature codes, edit the features and line-work in the survey, and access the powerful Road Link and DTM Link software.

4.1.3 Main Parts Of Geomatics Office

1. Tool Bar puts common actions just a mouse click away
2. Intuitive Project bar makes trimble geomatics office exceptionally easy to learn and use.
3. Click on an observation in the Distinct Colors allow different observation types to be easily distinguished, such as postprocessor GPS, real-time GPS, conventional optical and digital level observations.
4. Zoom Navigator window always shows the entire project area, with an outline of the current view area for quick and easy navigation around the project
5. Familiar, standard Windows interface
6. Plan View displays the survey plotted as features
7. Survey View displays the individual observations in the survey
8. Click on an observation in the survey view or on a feature in the plan view and the properties box displays the detailed information for that observation or feature.

4.1.4 Integrated Surveying

Trimble Geomatics Office brings all your survey data together. Integrated surveying is the key to fast and efficient surveying. The ability to seamlessly integrate a wide range of different survey data types is at the very heart of the Trimble Geomatics Office software. The software offers the freedom to use any survey instrument required for the job a survey-grade GPS receiver. Designed to handle every type of survey data that the surveyor and engineer are ever likely to use, the software has unrivalled data integration capabilities and can read in data such as:

- Real-time Kinematic (RTK) GPS data
- Raw GPS data.

- Road design data in more than 20 different native formats used worldwide, including: Autodesk Civil Design.
- Custom ASCII data

The software supports a two way flow of digital data, allowing data to be loaded into the TSC or TSC1 controllers running Trimble Survey Controller software for use in the field, such as:

- Points of all kinds, including control points, data points and design points.
- Datum and projection parameters.
- Digital terrain models (Grid or TIN).
- Geoids models.
- Combined datum grid files.
- Road design files complete with full alignment geometry.
- Feature and attribute libraries.
- GIS data dictionary files.

Data can also be exported in any one of over 30 data formats for third-party survey, design, CAD and GIS software including AutoCAD, Arc View, MapInfo, Micro station and many more.

4.1.5 GPS Baseline Processing and Network Adjustment.

The WAVE Baseline Processing and Network Adjustment modules in the Trimble Geomatics Office software are designed to be simplicity itself for those who just need the right answers fast while advanced controls can be accessed for those who need more control over the processing of their data. Intelligent default parameter values ensure that users who only want to press a button and see the right answers fast can do just that.

For the advanced user, advanced controls can be accessed using styles, allowing the user to take control of baseline processing parameters or network adjustment parameters. Extensive QC tools provide fast and accurate assessment of data quality, while visual cues, such as red flags, instantly alert the user to out of tolerance data. Data can also be browsed and queried visually using the graphical Timeline display.

The fast and powerful least-squares Network Adjustment module is also accessed from the survey view. Vectors can be included or excluded from the can be included or excluded from the adjustment with the click of the mouse. and observations can be queried or disabled graphically using simple point-and-click techniques. Combining GPS and conventional data in a network adjustment is simple; you get the results you need with identical workflows, no matter what data you have in the project.

After adjustment, each station's horizontal and vertical error ellipses are displayed in the survey view for quick and easy inspection of network quality. Fixing control points for the network adjustment is also quick and easy, enabling you to achieve the high quality results you need fast.

4.1.5 Quick and Simple CAD Work In The Plan View

Tidying up the survey for delivery to the client is quick and simple using the Trimble Geomatics Office software. Maximum use has been made of toolbars, graphics and the mouse commands to make the software exceptionally intuitive and easy to learn. Powerful feature code processing quickly transforms raw survey observations into a final plan. The software allows the user to define and customize feature codes, point and line styles, and other CAD elements.

4.1.7 Processing Road Design Data is fast and Easy

Using the innovative Road Link software, engineers and surveyors alike can now take just about any road design out into the field and perform stakeout with ease. The Trimble Geomatics Office software is designed to read in almost any type of road design data that you need to work with. Importing road design data is so easy It gives you full control of how the road design is imported. You can even import cross-section data to use as templates. Design files in the following native formats among others can be imported directly into the Trimble Geomatics Office software: Autodesk Civil Design.

Design data can be graphically displayed, edited, and then loaded into the Trimble Survey Controller software as a Job file for stakeout on site. The file allows the horizontal and vertical alignments and templates not just a list of points to be taken into the field. This enables any station and offset to be specified for stakeout, providing the field crew with the flexibility required in the construction environment.

When stakeout is complete, the Job file with as staked positions can be transferred back to the Trimble Geomatics Office software for a quality control check. The staked points are shown on a plan of the site and cut sheets can be prepared in a wide variety of cut sheet report formats to meet your needs.

4.1.8 Creating contour surface models and volume calculations

If you already have a contour model in the popular 3D Faces AutoCAD format, you can import it to DTM Link, as well as export any surface created in DTM Link in the3D faces format. Sending digital terrain models to the Trimble Survey Controller software for stakeout is simple, and you have the choice of using a gridded DTM or a Triangulated Irregular Network (TIN).

4.1.9 Outstanding Quality Assurance and Quality Control Capabilities

Quality control of survey data is more important today than it has ever been. With outstanding QA/QC capabilities you can be sure that your surveys are always produced to the high standards your clients expect. Every module of the Trimble Geomatics Office software is loaded with features to help you maintain the highest standards of quality and quickly spot problem data.

The time line window provides a graphical display of observed data over time. Customizable displays allow the advanced user to analyze different combinations of correlated variables for quality control or troubleshooting purposes. Timeline can be used to display dependent baselines, providing both the flexibility to determine independent baselines and full control of what is passed to a network adjustment just another way to help you process the data you need.

To check the quality of your post processed network, simply use the Loop Closures report. You have the choice of letting the Trimble Geomatics Office software perform the report either on the entire network or on a selection you define by using any of the extensive selection tools available.

For first time users and for those who just need their results fast the Trimble Geomatics Office software makes the maximum use of intelligent default values and graphical tools, such as the ability to graphically disable and edit observations for quick and painless completion of high quality surveys.

4.1.10 Minimum Hardware Requirements

The minimum requirements for the computer running the Trimble Geomatics Office software are:

- Pentium-based computer, 150 MHz or faster with 32- MB RAM and a 1-GB hard drive
- SVGA color 800 x 600
- Keyboard with mouse or trackball
- CD ROM drive

The Trimble Geomatics Office software operates under Microsoft Windows 95/98 /Me 2000/xp.

4.2 5700 GPS Receiver

The 5700 GPS receiver, which is designed for GPS surveying applications The 5700 receiver tracks GPS satellites on both the L1 and L2 frequencies to provide precise position data for land survey applications.

4.2.1 Parts of the Receiver

All operating controls, ports, and connectors on the 5700 receiver are located on its four main panels, as shown in Figure 4.2.

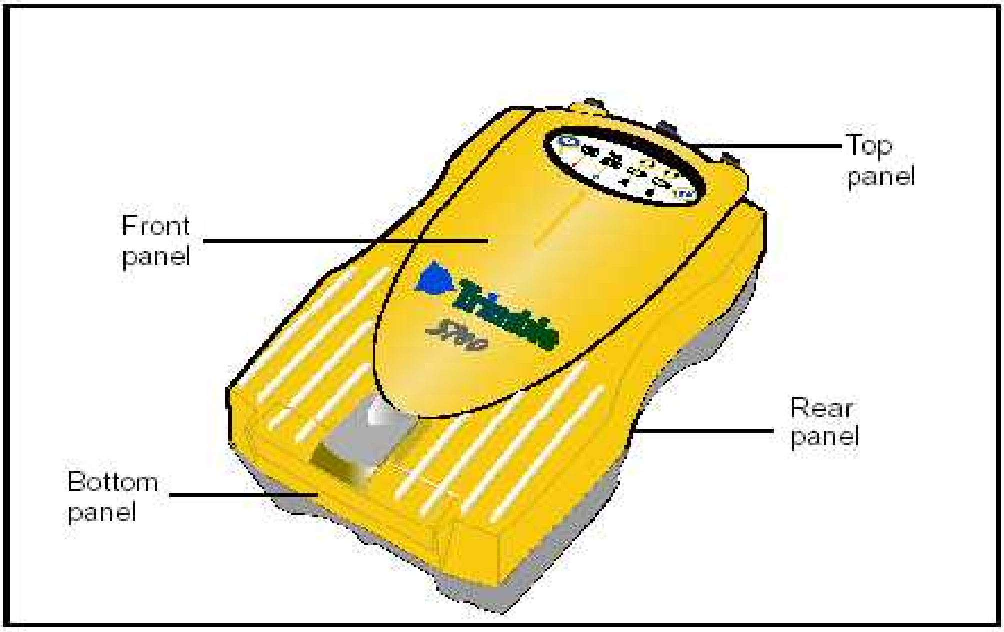


Figure 4.1: Panels on the 5700 receiver[1]

4.2.2 Setup Guidelines

Consider the following guidelines when setting up the 5700 receiver.

- **Environmental conditions**

Although the 5700 receiver has a waterproof housing, reasonable care should be taken to keep the unit dry. Avoid exposure to extreme environmental conditions, including:

- Water
- Heat greater than 65° C (149° F)
- Cold less than –40° C (–40° F)
- Corrosive fluids and gases

Avoiding these conditions improves the 5700 receiver's performance and long-term reliability.

[1] From Reference No.11

Sources of electrical interference

Avoid the following sources of electrical and magnetic noise:

- Gasoline engines (spark plugs).
- Televisions and PC monitors.
- Alternators and generators.
- Electric motors.
- Equipment with DC-to-AC converters.
- Fluorescent lights.
- Switching power supplies.

4.2.3 Post processed Setup

For a post processed survey, you only need:

- The 5700 receiver
- A Zephyr Geodetic antenna
- A GPS antenna cable

Other equipment, as described below, is optional. To set up the 5700 receiver for a post processed survey. Set up the tripod with the tribrach and antenna adapter over the survey mark. Instead of a tripod, you can use a range pole with a bipod. However, Trimble recommends that you use a tripod for greater stability.

1. Mount the antenna on the tribrach adapter.
2. Use the tripod clip to hang the 5700 receiver on the tripod.
3. Connect the yellow GPS antenna cable to the Zephyr antenna.
4. Connect the other end of the GPS antenna.

Figure 4.2: Post processed setup[1]

4.2.4 Other System Components

- **Radios**

Radios are the most common data link for Real-Time Kinematic(RTK) surveying. The 5700 receiver is available with an optional internal radio in either the 450 or 900 MHz UHF bands.

- **Antennas**

The 5700 receiver should normally be used with a Zephyr or Zephyr Geodetic antenna. These antennas have been designed specifically for use with the 5700 receiver. Use Figure 4.3 as a guide for measuring the height of the Zephyr and Zephyr Geodetic antennas. The Zephyr antenna is designed to be measured to the top of the notch. The Zephyr Geodetic (shown) has been designed to be measured to the bottom of the notch.

[1] From Reference No.11

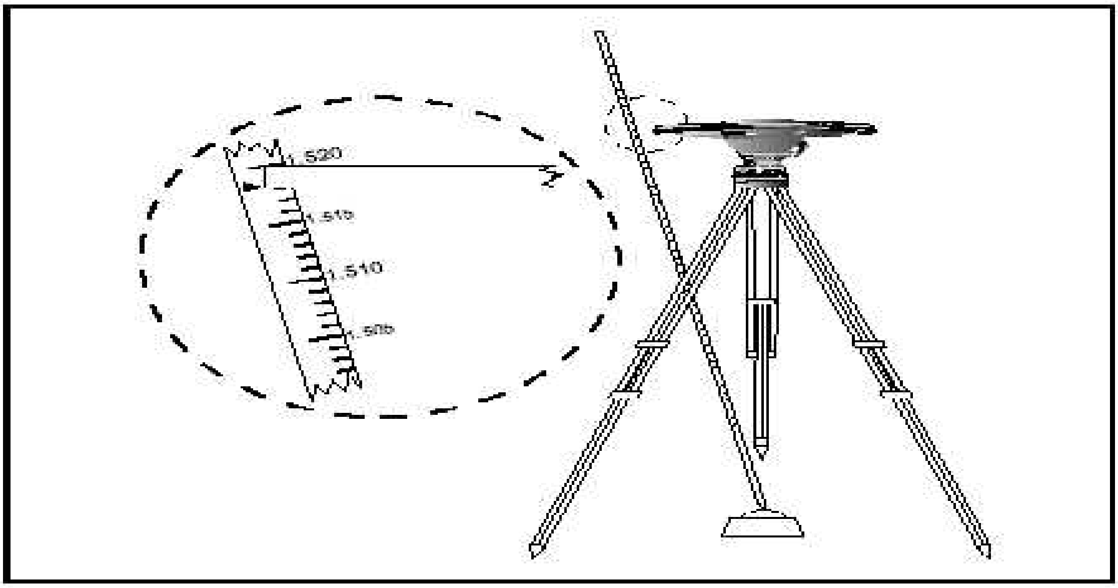


Figure 4.3: Measuring antenna height[1]

4.3 Setup receiver and Start RTK Survey and Static.

For a post processed survey:

- Using 5700 receiver
- A Zephyr™ or Zephyr Geodetic antenna(trade name)
- AGPS antenna cable
- Set up the tripod with the tribrach and antenna adapter over the survey mark.
- Use the tripod clip to hang the 5700 receiver on the tripod.
- Connect the yellow GPS antenna cable to the Zephyr antenna.
- Connect the other end of the GPS antenna cable to the yellow port on the 5700 receiver.
- connect a battery
- Place the bracket against the pole at a comfortable height.
- Rotate the clamping screw on the bracket pole until tight.
- Place the controller into the cradle assembly and tighten the clamping mechanism.
- Any cables running down the pole should be run through the machined groove on the inside of the controller bracket.

---

[1] From Reference No.11

- Position the controller in the preferred position for operation by pressing the spring-loaded release button on the cradle, pulling the assembly outward, and rotating the cradle assembly to the desired angle.
- Connect the other end of the cable to Port on the 5700 receiver.
- Use the On/Off button  to switch the receiver.
- Turn on the controller Survey.
- From the main menu on the controller, select *Configuration*.
- In the dialog that appears, select *Survey styles*, then from the
- *Type* field, select *RTK, select start base receiver ,Palestinian old gird ,name point ,code, coordinate the base and antenna height*.
- Start observation and store.

On returning to the office after completing a survey, transfer the field data to a computer that has the Trimble Geomatics Office™ software installed. You can then process the survey data in Trimble Geomatics Office to produce baselines and coordinates.

- Use the USB cable to connect the 5700 receiver to the computer.
- Start Trimble Geomatics Office.
- Transfer the data files to the computer using the Trimble Data Transfer utility.

Create a project:

- Select File */New Project.*
- In the *Name* field of the dialog that appears, enter a name for the Project.
- From the *Template* list, select the *Sample data* option.
- In the *New* group, make sure that the *Project* option is selected and then click **ok.**
- The Project is created and the *Project Properties* dialog appears.
- The values in the fields in each tab are derived from the sample data template.
- To close the *Project Properties* dialog, click **ok.**

CHAPTER FIVE

FIELD MEASUREMENTS

This chapter includes field measurements using "GPS".

5.1 Introduction

5.2 Positioning of Trimble's main antenna

5.3 Network establishment

5.4 Network measurements

5.5 RTK observations

5.6 Static observation

5.7 Data Processing and Analysis

CHAPTER FIVE

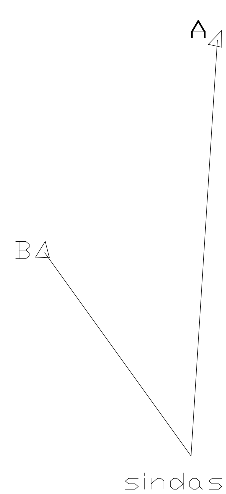
FIELD MEASUREMENTS

5-1 Introduction

In this section, the measurements for this Project are outlined. The outgoing task for this work was to compare RTK; static to be able to do this a control network had to be established. This control network had to be placed nearby the PPU University, to get a good connection between the rover and the base on the roof Building A and B. The idea was to, by relative positioning, have one permanent GPS- receiver on the roof continuously collecting data from satellites.

5.2 Positioning of Trimble's main antenna

Tow well defined points was used to decide the position of Trimble's antenna in *Palestinian old gird* reference system. The positioning was done using static observations and post-processing. For every static observation, the Trimble 5700 was used and the observation time for each baseline was three hours. The coordinates calculated for the points were in *Palestinian old gird* and later used to fix the points before the net adjustment in the Least Squar



A: Control Point on Building A
B: Control Point on Building B

Figure 5.1: Baselines from the known points to the roof point.

All result was done using the software "Trimble Geomatics Office". The result is presented in chapter six.

5.3 Network establishment

This section describes the procedure of positioning the points in the frame network. The instruments used Trimble 5700.

The main points in this network were positioned using static observations and each baseline was observed for more than there hours, Figure 5.2 illustrates GPS network, in this figure, station A and B are control station, and stations C, D, E, and F are points of unknown position.

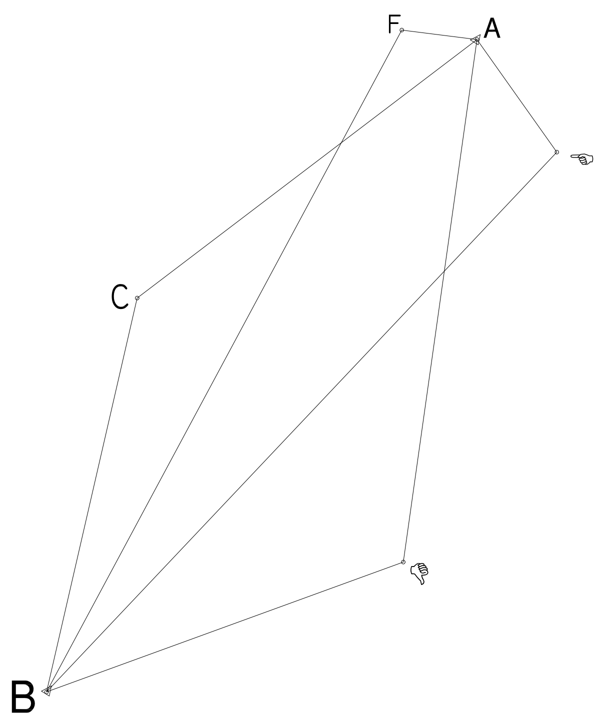


Figure 5.2: Proposed GPS Survey network.

The first milestone in the project was to establish a network of control points inside the PPU Campus, at southern part of Hebron city. After positioning the roof point in the earlier measurements, the control points could be determined with normal static GPS-observations and RTK relative to the roof point (A&B).

5.4 Network measurements

The antenna on the roof (A&B) was positioned using three hours static GPS observation, from four well-defined points in the nearby region. The coordinates were in the *Palestinian old gird* system. The points were located in C, E, D, and F. A 5700 rover collected data on each of the four points 5700 receiver, with a Zephyr Geodetic antenna , simultaneously collected data throughout the measurements on the roof point.

5.5 RTK observations

The network points were observed with normal Radio RTK, with three different time intervals: 10 seconds, 1 minute, and 5 minute. These measurements were done in relation to the base station on the roof. All the time intervals were observed separately, with no correlation to each other. This way the time aspect was easier to include in the comparison. During the observations, the times for all events were noted. This way the decision of what to compare could be done later on.

All measurements were done, simulating a normal Radio-RTK job, where ten points should be positioned. The reason for measuring at three different time intervals was to analyze if a longer observation time improves the accuracy. The coordinates for each points and each interval, was later on compared separately with the coordinates from the static observations.

5.6 Static observation

The only difference from ordinary Radio- RTK is how the correction data, from the base and the rover, is transferred. Radio- RTK uses a radio link and static stored in base and rover connection data in TGF "Trimble Geomatics Office" transfer in computer.

Every control point was exactly as the Network observations, measured in three different time epochs 8minutes, 15minutes and 30minutes. The measuring epochs were made independently from the others. That way comparison between the intervals could easily be done with no correlations between the measurements.

5.7 Data Processing and Analysis

5.7.1 Specification for GPS Surveys (Order of accuracy)

The” Geometric Geodetic Accuracy Standards and specification for using GPS Relative Positioning Techniques” specifies seven different orders relative positioning in Table 5.1

Order	Allowable error Ratio	Parts-Per-Million (PPM)
AA	1:100,000,000	0,01
A	1:10,000,000	0,1
B	1:1,000,000	1,0
C-1	1:100,000	10
C-2-I	1:50,000	20
C-2-II	1:20,000	50
C-3	1:10,000	100

5.7.2 Analysis of Fixed Baseline Measurements

The GPS job specification often requires that baseline measurements be taken between base stations; these measurements can be used to confirm the accuracy of both the GPS measurement system and the known baseline lengths.

If we assume that we have the X, Y and Z coordinates to the base stations A and B (measured by the electronic distance measurement – EDM instruments, with a high accuracy), then we can compute (X_{AB}, Y_{AB}, Z_{AB}) and then the procedure can be:

1- Computing the coordinate differences between the 2 measured (using GPS) points (Δx_{AB}, Δy_{AB}, Δz_{AB}).

2- Computing the absolute values of differences between the measured and the known baselines ($dX = \Delta X_{AB} - \Delta x_{AB}$, $dY = \Delta Y_{AB} - \Delta y_{AB}$, $dZ = \Delta Z_{AB} - \Delta z_{AB}$).

3- Computing the length of the base line as:

$$AB = \sqrt{[(\Delta X_{AB})^2 + (\Delta Y_{AB})^2 + (\Delta Z_{AB})^2} . \quad (5.1)$$

4- Express the differences as computed in step 2 in parts per million, by dividing the Differences by the length of the baseline computed in step 3; as:

$$\Delta X\text{-ppm} = dX/AB*1,000,000$$
$$\Delta y\text{-ppm} = dY/AB*1,000,000 \quad (5.2)$$
$$\Delta z\text{-ppm} = dZ/AB*1,000,000$$

5- Check the computed values for the ppm with the standard values.

5.7.3 Analysis of Repeat Baseline Measurements

Making repeated measurements of certain baselines can do removing blunders. The repeated measurements are taken in different observation sessions, and the results compared by finding the absolute differences in the two measurements and computing the ppm values.

5.7.4 Analysis of Loop Closures

GPS surveys consist of many interconnected closed loops, which form a network. For each closed loop, the algebraic sum of the X components should equal zero, the same condition should exist also for the Y and Z components, Large closing values within any closed loop indicate that either a blunder or a large error exist in one or more of the baselines of the loop.

It is important not to include any trivial baselines in these computations, because they can

Produce false accuracies for the loop. To compute loop closing, the baseline components are added algebraically for that loop ACBDEA as;

$CX = \Delta XAC + \Delta XCB + \Delta XBD + \Delta XDE + \Delta XEA$

$CY = \Delta YAC + \Delta YCB + \Delta YBD + \Delta YDE + \Delta YEA$

$CZ = \Delta ZAC + \Delta ZCB + \Delta ZBD + \Delta ZDE + \Delta ZEA$

To compute the total length (lc) of the disclosure:

$$lc = \sqrt{(CX)^2 + (CY)^2 + (CZ)^2} \qquad (5.3)$$

And then we can find the closure ppm ratio by dividing CX, CY, CZ by lc and then compare them with the standard values.

5.7.5 Least-squares Adjustment of GPS Networks.

As noted earlier, because GPS network contain redundant measurements, they must be adjusted to make all coordinate difference consistent. In applying least squares to the problem of Adjustment baselines in GPS networks, observation equations are written that relate station coordinates to observed coordinate differences and their residual errors, to illustrate the procedure in Figure 5.2 for line AC of this figure an observation equation can be written for each measured baseline component as:

$$X_C = X_A + \Delta X_{AC} + V_{X\,AC}$$

$$Y_C = Y_A + \Delta Y_{AC} + V_{Y\,AC} \qquad (5.4)$$

$$Z_C = Z_A + \Delta Z_{AC} + V_{Z\,AC}$$

For line AE.

$X_E = X_A + \Delta X_{AE} + V_{XAE}$

$$Y_E = Y_A + \Delta Y_{AE} + V_{YAE} \quad (5.5)$$

$$Z_E = Z_A + \Delta Z_{AE} + V_{ZAE}$$

For line AF:

$$X_F = X_A + \Delta X_{AF} + V_{XAF}$$

$$Y_F = Y_A + \Delta Y_{AF} + V_{YAF} \quad (5.6)$$

$$Z_F = Z_A + \Delta Z_{AF} + V_{ZAF}$$

For line AD:

$$X_D = X_A + \Delta X_{AD} + V_{XAD}$$

$$Y_D = Y_A + \Delta Y_{AD} + V_{YAD} \quad (5.7)$$

$$Z_D = Z_A + \Delta Z_{AD} + V_{ZAD}$$

For line BC:

$$X_C = X_B + \Delta X_{BC} + V_{XBC}$$

$$Y_C = Y_B + \Delta Y_{BC} + V_{YBC} \quad (5.8)$$

$$Z_C = Z_B + \Delta Z_{BC} + V_{ZBC}$$

For line BD:

$$X_D = X_B + \Delta X_{BD} + V_{XBD}$$

$$Y_D = Y_B + \Delta Y_{BD} + V_{YBD} \quad (5.9)$$

$$Z_D = Z_B + \Delta Z_{BD} + V_{ZBD}$$

For line BE:

$$X_E = X_B + \Delta X_{BE} + V_{XBE}$$

$$Y_E = Y_B + \Delta Y_{BE} + V_{YBE} \quad (5.10)$$

$Z_E = Z_B + \Delta Z_{BE} + V_{ZBE}$

for line BF:

$X_F = X_B + \Delta X_{BF} + V_{XBF}$

$Y_F = Y_B + \Delta Y_{BF} + V_{YBF}$ (5.11)

$Z_F = Z_B + \Delta Z_{BF} + V_{ZBF}$

Observation equation of the form above would be written for all measured baseline in any figure. For Figure 5.1 there were total of 8 measured baselines, so the number of observation equation that can be developed is 24.Also, each of station C, D, E, and F has three unknown coordinates, for a total of 12 unknown in the problem. Thus there is 24-12=12 redundant observation in the network. The 24 observation equation can be expressed in matrix form as:

$$AX=L+V \quad (5.12)$$

If the observation equation for adjusting the network of figure 5.2 are written in the same order that the measurement are listed in chapter six in table the A, X, L ,V ,and W matrices would be. The system of observation equation (5.12) is solved by least squares using Equation

$X= N^{-1} A^T W L.$

$N = (A^T W) A.$

Calculate the reference standard deviation for the adjustment using the matrix expression of Equation $S_0 = \sqrt{\frac{V^T WV}{r}}$ (5.13)

$r = m\text{-}n$, m is the number of observation and n is the number of unknown.

The estimated standard deviation $S_{i,}$ for any unknown parameter, having been computed from a system of observation equations, is expressed as

$S_i = S_0 \sqrt{Q_{Xi\,Xi}}$, where $Q_{xi\,xi}$ is the diagonal element (from the ith row and ith column) of The Q_{xx} matrix, $Q_{xx} = (A^T WA)^{-1}$.

$$\mathbf{A}=\begin{bmatrix}
1&0&0&0&0&0&0&0&0&0&0&0\\
0&1&0&0&0&0&0&0&0&0&0&0\\
0&0&1&0&0&0&0&0&0&0&0&0\\
0&0&0&1&0&0&0&0&0&0&0&0\\
0&0&0&0&1&0&0&0&0&0&0&0\\
0&0&0&0&0&1&0&0&0&0&0&0\\
0&0&0&0&0&0&1&0&0&0&0&0\\
0&0&0&0&0&0&0&1&0&0&0&0\\
0&0&0&0&0&0&0&0&1&0&0&0\\
0&0&0&0&0&0&0&0&0&1&0&0\\
0&0&0&0&0&0&0&0&0&0&1&0\\
0&0&0&0&0&0&0&0&0&0&0&1\\
1&0&0&0&0&0&0&0&0&0&0&0\\
0&1&0&0&0&0&0&0&0&0&0&0\\
0&0&1&0&0&0&0&0&0&0&0&0\\
0&0&0&1&0&0&0&0&0&0&0&0\\
0&0&0&0&1&0&0&0&0&0&0&0\\
0&0&0&0&0&1&0&0&0&0&0&0\\
0&0&0&0&0&0&1&0&0&0&0&0\\
0&0&0&0&0&0&0&1&0&0&0&0\\
0&0&0&0&0&0&0&0&1&0&0&0\\
0&0&0&0&0&0&0&0&0&1&0&0\\
0&0&0&0&0&0&0&0&0&0&1&0\\
0&0&0&0&0&0&0&0&0&0&0&1
\end{bmatrix}_{24\times 12}$$

$$\mathbf{X} = \begin{bmatrix} X_C \\ Y_C \\ Z_C \\ X_E \\ Y_E \\ Z_E \\ X_D \\ Y_D \\ Z_D \\ X_F \\ Y_F \\ Z_F \end{bmatrix}_{12\times 1} \qquad \mathbf{L} = \begin{bmatrix} X_A + \Delta X_{AC} \\ Y_A + \Delta Y_{AC} \\ Z_A + \Delta Z_{AC} \\ . \\ . \\ . \\ . \\ . \\ . \\ . \\ . \\ . \\ X_B + \Delta X_{BF} \\ Y_B + \Delta Y_{BF} \\ Z_B + \Delta Z_{BF} \end{bmatrix}_{24\times 1} \qquad \mathbf{V} = \begin{bmatrix} V_{XAC} \\ V_{YAC} \\ V_{ZAC} \\ . \\ . \\ . \\ . \\ . \\ . \\ . \\ . \\ . \\ V_{XBF} \\ V_{YBF} \\ V_{ZBF} \end{bmatrix}_{24\times 1}$$

$$
\mathbf{W} = \begin{bmatrix}
\frac{1}{(\pm_{XX})^2} & \frac{1}{(\pm_{XY})^2} & \frac{1}{(\pm_{XZ})^2} & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 & 0 \\
\frac{1}{(\pm_{XY})^2} & \frac{1}{(\pm_{XY})^2} & \frac{1}{(\pm_{YZ})^2} & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 & 0 \\
\frac{1}{(\pm_{ZX})^2} & \frac{1}{(\pm_{ZY})^2} & \frac{1}{(\pm_{ZZ})^2} & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & . & & & & & & . & . & . \\
0 & 0 & 0 & . & . & & & & & . & . & . \\
0 & 0 & 0 & . & & . & & & & . & . & . \\
\vdots & \vdots & \vdots & \vdots & & & \ddots & & & \vdots & \vdots & \vdots \\
. & . & . & . & & & & . & & 0 & 0 & 0 \\
. & . & . & . & & & & & . & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 & \frac{1}{(\pm_{XX}BF)^2} & \frac{1}{(\pm_{XY}BF)^2} & \frac{1}{(\pm_{XZ}BF)^2} \\
0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 & \frac{1}{(\pm_{XY}BF)^2} & \frac{1}{(\pm_{YY}BF)^2} & \frac{1}{(\pm_{YZ}BF)^2} \\
0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 & \frac{1}{(\pm_{ZX}BF)^2} & \frac{1}{(\pm_{YZ}BF)^2} & \frac{1}{(\pm_{ZZ}BF)^2}
\end{bmatrix}_{24 \times 24}
$$

CHAPTER SIX

CALCULATIONS

This chapter includes calculations using lest square solution.

6.1 Positioning of Trimble's main antenna.

6.2 Accuracy in Static.

6.3 Accuracy in RTK.

6.4 Mean Standard Deviation

6.5 Calculation best fit line.

6.6 Curves of static mean standard error.

6.7 Curves of RTK mean standard error.

6.8 General Formula Computed standard deviation.

6.9 Comparison.

CHAPTER SIX

CALCULATIONS

6.1 Positioning of Trimble's main antenna.

The result and data from the positioning of Trimble's main antenna is shown in the subsections below.

6.1.1 Known coordinates

Table 6.1: Known coordinates.

Point	Northing(Y)m	Easting(X)m	Elevation(Z)m
A	101825.76949851	158809.741073894	915.9934587
B	101500.7686958	158586.046678492	916.4025426
Sindas[1]	101189.4899	158775.6199	921.6432

These are coordinates received from trig sindas. The coordinates are for our purposes considered error-free and fixed before the net-adjustment in lest square.

6.1.2 Time.

Table 6.2: Time

Point	Measuring date	GPS-time(hours)	Antenna height(m)
A	10-07-2005	12:30 – 15:30	1.65
B	10-09 -2005	09:30 – 12:30	1.70

[1] Certified from the surveying Department

The observations are made with a Trimble 5700 receiver, and the antenna heights are measured to the bottom of the antenna mount. Appendix B Show Static and RTK observation respectively.

After applying Least Squares (see appendix A) Computed Reference Variance at time 8 Minutes $V^TWV = 115.7299438$.

Computed S0 = $\sqrt{\frac{V^TWV}{r}}$, (r) Degrees of Freedom =*12* , S0 = 3.105505109.

Computed Standard Deviation 8 Minutes in Station A.

$S_{XC} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{2.3769186116\text{E}-06}$ = 4.7878375190E-03

$S_{YC} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{1.5409481697\text{E}-06}$ = 3.85501670E-03

$S_{ZC} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{1.8199623608\text{E}-06}$ =4.18951212065E-03

$S_{XD} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{2.2581067819\text{E}-06}$ = 4.6666420023E-03

$S_{YD} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{1.6001657531\text{E}-06}$ = 3.92839123829E-03

$S_{ZD} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{2.4083634765\text{E}-06}$ = 4.81940322847E-03

$S_{XE} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{4.7931346223\text{E}-06}$ =6.7989533532E-03

$S_{YE} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{3.4063028872\text{E}-06}$ =5.73157367607E-03

$S_{ZE} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{4.9994896846\text{E}-06}$ =6.94376614999E-03

$S_{XF} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{2.9037449304\text{E}-06}$ =5.2918981909E-03

$S_{YF} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{2.0064076711\text{E}-06}$ =4.39887719560E-03

$S_{ZF} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{2.8300411149\text{E}-06}$ =5.22430616713E-03

Table 6.3: Standard Deviation 8 Minutes in Station A.

Standard Deviation 8 Minutes in Station A				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	3.85501670E-03	4.7878375190E-03	4.18951212065E-03
2	**D**	3.92839124E-03	4.6666420023E-03	4.81940322847E-03
3	**E**	5.73157368E-03	6.7989533532E-03	6.94376614999E-03
4	**F**	4.39887720E-03	5.2918981909E-03	5.22430616713E-03

Computed Standard Deviation 8 Minutes in Station B.

$S_{XC} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{1.9912907803\text{E}-06}$ = 4.38227461926E-03

$S_{YC} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{1.4106501768\text{E}-06}$ = 3.68843310972E-03

$S_{ZC} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{2.1215762194\text{E}-06}$ =4.52336431397E-03

$S_{XD} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{1.9577646323\text{E}-06}$ = 4.34522717895E-03

$S_{YD} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{1.1137277634\text{E}-06}$ = 3.27734205629E-03

$S_{ZD} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{1.0828417058\text{E}-06}$ = 3.23157868724E-03

$S_{XE} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{2.2211956132\text{E}-06}$ =4.62834422744E-03

$S_{YE} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{1.3656875889\text{E}-06}$ =3.62917515759E-03

$S_{ZE} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{1.5445458785\text{E}-06}$ =3.85951430119E-03

$S_{XF} = S_0\sqrt{Q_{Xi\,Xi}}$ = 3.105505109 $\sqrt{1.7542010350\text{E}-06}$ =4.11312520242E-03

$S_{YF} = S_0\sqrt{Q_{Yi\,Yi}}$ = 3.105505109 $\sqrt{1.1270385273\text{E}-06}$ = 3.29686853200E-03

$S_{ZF} = S_0\sqrt{Q_{Zi\,Zi}}$ =3.105505109 $\sqrt{1.3985855349\text{E}-06}$ =3.67262650488E-03

Table 6.4: Standard Deviation 8 Minutes in Station B

Standard Deviation 8 Minutes in Station B				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	3.68843311E-03	4.3822746193E-03	4.52336431397E-03
2	**D**	3.27734206E-03	4.3452271790E-03	3.23157868724E-03
3	**E**	3.62917516E-03	4.6283442274E-03	3.85951430119E-03
4	**F**	3.29686853E-03	4.1131252024E-03	3.67262650488E-03

Computed Average standard Deviation 8 Minutes in Station A and B.

Average standard Deviation at point = $\frac{\text{standard Deviation A} + \text{standard Deviation B}}{2}$

$S_{XC}= \frac{4.7878375190E-03 + 4.38227461926E-03}{2}$ =4.5850560691E-03

$S_{YC}= \frac{3.85501670E-03 + 3.68843310972E-03}{2}$ =3.7717249047E-03

$S_{ZC}= \frac{4.18951212065E-03 + 4.52336431397E-03}{2}$ =4.3564382173E-03

$S_{XD}= \frac{4.6666420023E-03 + 4.34522717895E-03}{2}$ =4.5059345906E-03

$S_{YD}= \frac{3.92839123829E-03 + 3.27734205629E-03}{2}$ =3.6028666473E-03

$S_{ZD}= \frac{4.81940322847E-03 + 3.23157868724E-03}{2}$ =4.0254909579E-03

$S_{XE}= \frac{6.7989533532E-03 + 4.62834422744E-03}{2}$ =5.7136487903E-03

$S_{YE}=\frac{5.73157367607E-03+3.62917515759E-03}{2}$ =4.6803744168E-03

$S_{ZE}=\frac{6.94376614999E-03+3.85951430119E-03}{2}$ =5.4016402256E-03

$S_{XF}=\frac{5.2918981909E-03+4.11312520242E-03}{2}$ =4.7025116967E-03

$S_{YF}=\frac{4.39887719560E-03+3.29686853200E-03}{2}$ =3.8478728638E-03

$S_{ZF}=\frac{5.22430616713E-03+3.67262650488E-03}{2}$ =4.4484663360E-03

Table 6.5: Average Standard Deviation 8 Minutes.

Average Standard Deviation 8 Minutes.				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	3.7717249047E-03	4.5850560691E-03	4.3564382173E-03
2	**D**	3.6028666473E-03	4.5059345906E-03	4.0254909579E-03
3	**E**	4.6803744168E-03	5.7136487903E-03	5.4016402256E-03
4	**F**	3.8478728638E-03	4.7025116967E-03	4.4484663360E-03

Computed Horizontal Standard Error the point C, D, E and F at the time 8 Minutes.

Horizontal Standard Error $=\sqrt{(S_X)^2+(S_Y)^2}$

Horizontal Standard $_C=\sqrt{(4.5850560691E-03)^2+(3.7717249047E-03)^2}$ = 0.005937057

Horizontal Standard $_D=\sqrt{(4.5059345906E-03)^2+(3.6028666473E-03)^2}$ = 0.005769237

Horizontal Standard $_E=\sqrt{(5.7136487903E-03)^2+(4.6803744168E-03)^2}$ =0.007385911

Horizontal Standard $_F=\sqrt{(4.7025116967E-03)^2+(3.8478728638E-03)^2}$ = 0.006076162

Computed Vertical Standard Error the point C, D, E and F at the time 8 Minutes.

Vertical Standard Error = $\sqrt{(S_Z)^2}$

Vertical Standard Error $_C$ = $\sqrt{(4.3564382173E-03)^2}$ =4.3564382173E-03

Vertical Standard Error $_D$= $\sqrt{(4.0254909579E-03\ )^2}$ =4.0254909579E-03

Vertical Standard Error $_E$ = $\sqrt{(5.4016402256E-03)^2}$ =5.4016402256E-03

Vertical Standard Error $_F$ = $\sqrt{(4.4484663360E-03)^2}$ =4.4484663360E-03

Table 6.6: Horizontal and Vertical Standard Error in 8 Minutes.

Horizontal and Vertical Standard Error in 8 Minutes.			
Point	**Name**	**Horizontal**	**Vertical**
1	**C**	0.005937057	4.3564382173E-03
2	**D**	0.005769237	4.0254909579E-03
3	**E**	0.007385911	5.4016402256E-03
4	**F**	0.006076162	4.4484663360E-03

After applying Least Squares(see appendix A) Computed Reference Variance at time 15 Minutes $V^TWV = 0.000197987$ **.**

$S_0 = \sqrt{\frac{V^TWV}{r}}$ **,** (r) Degrees of Freedom =*12* , S_0 = 0.004061887

Computed Standard Deviation 15 Minutes in Station A.

$S_{XC} = S_0\sqrt{Q_{Xi\,Xi}}$ = 0.004061887 $\sqrt{1.31010210229E-06}$ =4.6492236849E-06

$S_{YC} = S_0\sqrt{Q_{Yi\,Yi}}$ = 0.004061887 $\sqrt{8.5156552454E-7}$ = 3.7483221147E-06

$S_{ZC} = S_0\sqrt{Q_{Zi\,Zi}}$ $=0.004061887\sqrt{1.06447752414E-06}$ $=4.1907921122E-06$

$S_{XD} = S_0\sqrt{Q_{Xi\,Xi}}$ $= 0.004061887\sqrt{2.69588229818E-06}$ $= 6.6692705147E-06$

$S_{YD} = S_0\sqrt{Q_{Yi\,Yi}}$ $= 0.004061887\sqrt{1.9122439927E-06}$ $= 5.6169366073E-06$

$S_{ZD} = S_0\sqrt{Q_{Zi\,Zi}}$ $=0.004061887\sqrt{2.86348132434E-06}$ $= 7.0816478340E-06$

$S_{XE} = S_0\sqrt{Q_{Xi\,Xi}}$ $= 0.004061887\sqrt{2.99267690998E-06}$ $=7.0268031328E-06$

$S_{YE} = S_0\sqrt{Q_{Yi\,Yi}}$ $= 0.004061887\sqrt{2.1048947351E-06}$ $=5.8930898236E-06$

$S_{ZE} = S_0\sqrt{Q_{Zi\,Zi}}$ $=0.004061887\sqrt{3.03957533606E-06}$ $=7.0816478340E-06$

$S_{XF} = S_0\sqrt{Q_{Xi\,Xi}}$ $= 0.004061887\sqrt{2.25169028174E-06}$ $=6.0951191040E-06$

$S_{YF} = S_0\sqrt{Q_{Yi\,Yi}}$ $= 0.004061887\sqrt{1.9431314815E-06}$ $= 5.6621186245E-06$

$S_{ZF} = S_0\sqrt{Q_{Zi\,Zi}}$ $=0.004061887\sqrt{2.39617412322E-06}$ $=6.2876311483E-06$

Table 6.7: Standard Deviation 15 Minutes in Station A.

Standard Deviation 15 Minutes in Station A				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	3.74832211E-06	4.64922E-06	4.19079211216E-06
2	**D**	5.61693661E-06	6.6692705147E-06	7.08164783399E-06
3	**E**	5.89308982E-06	7.0268031328E-06	7.08164783399E-06
4	**F**	5.66211862E-06	6.0951191040E-06	6.28763114833E-06

Computed Standard Deviation 15 Minutes in Station B.

$S_{XC} = S_0\sqrt{Q_{Xi\,Xi}}$ $= 0.004061887\sqrt{4.70311766452E-06}$ $= 8.8088819907E-06$

$S_{YC} = S_0 \sqrt{Q_{Yi\,Yi}}$ = 0.004061887 $\sqrt{1.4106501768E-06}$ = 4.82433585E-06

$S_{ZC} = S_0 \sqrt{Q_{Zi\,Zi}}$ =0.004061887 $\sqrt{1.85531660434E-06}$ =5.53269698626E-06

$S_{XD} = S_0 \sqrt{Q_{Xi\,Xi}}$ = 0.004061887 $\sqrt{5.30005011001E-06}$ = 9.3512110256E-06

$S_{YD} = S_0 \sqrt{Q_{Yi\,Yi}}$ = 0.004061887 $\sqrt{8.9688655431E-06}$ = 1.21645662E-05

$S_{ZD} = S_0 \sqrt{Q_{Zi\,Zi}}$ =0.004061887 $\sqrt{1.27079387043E-05}$ = 1.44798954301E-05

$S_{XE} = S_0 \sqrt{Q_{Xi\,Xi}}$ = 0.004061887 $\sqrt{4.50221337094E-06}$ =8.6186829828E-06

$S_{YE} = S_0 \sqrt{Q_{Yi\,Yi}}$ = 0.004061887 $\sqrt{4.6629370706E-06}$ =8.77117239E-06

$S_{ZE} = S_0 \sqrt{Q_{Zi\,Zi}}$ =0.004061887 $\sqrt{3.53208254684E-06}$ =7.63384420104E-06

$S_{XF} = S_0 \sqrt{Q_{Xi\,Xi}}$ = 0.004061887 $\sqrt{6.14713919323E-06}$ =1.0070809790E-05

$S_{YF} = S_0 \sqrt{Q_{Yi\,Yi}}$ = 0.004061887 $\sqrt{4.3096981556E-06}$ = 8.43240189E-06

$S_{ZF} = S_0 \sqrt{Q_{Zi\,Zi}}$ =0.004061887 $\sqrt{3.13888374514E-06}$ =7.19640315592E-06

Table 6.8: Standard Deviation 15 Minutes in Station B.

Standard Deviation 15 Minutes in Station B				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	4.82433585E-06	8.8088819907E-06	5.53269698626E-06
2	**D**	1.21645662E-05	9.3512110256E-06	1.44798954301E-05
3	**E**	8.77117239E-06	8.6186829828E-06	7.63384420104E-06
4	**F**	8.43240189E-06	1.0070809790E-05	7.19640315592E-06

Computed Average standard Deviation at 15 Minutes in Station A and B.

Average standard Deviation at point **=** $\dfrac{\text{standard Deviation A} + \text{standard Deviation B}}{2}$

$$S_{XC}=\frac{4.6492236849E-06+8.8088819907E-06}{2}=6.7290528378E-06$$

$$S_{YC}=\frac{3.7483221147E-06+4.824335846626E-06}{2}=4.2863289807E-06$$

$$S_{ZC}=\frac{4.1907921122E-06+5.532696986256E-06}{2}=4.8617445492E-06$$

$$S_{XD}=\frac{6.6692705147E-06+9.351211025606E-06}{2}=8.0102407701E-06$$

$$S_{YD}=\frac{5.6169366073E-06+1.216456620851E-05}{2}=8.8907514079E-06$$

$$S_{ZD}=\frac{7.0816478340E-06+1.447989543008E-05}{2}=1.0780771632E-05$$

$$S_{XE}=\frac{7.0268031328E-06+8.618682982787E-06}{2}=7.8227430578E-06$$

$$S_{YE}=\frac{5.8930898236E-06+8.771172394922E-06}{2}=7.3321311093E-06$$

$$S_{ZE}=\frac{7.0816478340E-06+7.633844201040E-06}{2}=7.3577460175E-06$$

$$S_{XF}=\frac{6.0951191040E-06+1.007080979048E-05}{2}=8.0829644472E-06$$

$$S_{YF}=\frac{5.6621186245E-06+8.432401887441E-06}{2}=7.0472602560E-06$$

$$S_{ZF}=\frac{6.2876311483E-06+7.196403155923E-06}{2}=6.7420171521E-06$$

Table 6.9: Average Standard Deviation 15 Minutes .

Standard Average Deviation 15 Minutes.				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	4.2863289807E-06	6.7290528378E-06	4.8617445492E-06
2	**D**	8.8907514079E-06	8.0102407701E-06	1.0780771632E-05
3	**E**	7.3321311093E-06	7.8227430578E-06	7.3577460175E-06
4	**F**	7.0472602560E-06	8.0829644472E-06	6.7420171521E-06

Computed Horizontal Standard Error the point C, D, E and F at the time 15 Minutes.

Horizontal Standard Error $= \sqrt{(S_X)^2 + (S_Y)^2}$

Horizontal Standard $_C = \sqrt{(6.7290528378\text{E} - 06)^2 + (4.2863289807\text{E} - 06)^2}$ = 7.97827E-06

Horizontal Standard $_D = \sqrt{(8.0102407701\text{E} - 06)^2 + (8.8907514079\text{E} - 06)^2}$ = 1.1967E-05

Horizontal Standard $_E = \sqrt{(7.8227430578\text{E} - 06)^2 + (7.3321311093\text{E} - 06)^2}$ = 1.07217E-05

Horizontal Standard $_F = \sqrt{(8.0829644472\text{E} - 06)^2 + (7.0472602560\text{E} - 06)^2}$ = 1.07237E-05

Computed Vertical Standard Error the point C, D, E and F at the time 15 Minutes.

Vertical Standard Error $= \sqrt{(S_Z)^2}$

Vertical Standard Error $_C = \sqrt{(4.8617445492\text{E} - 06)^2}$ =4.8617445492E-06

Vertical Standard Error $_D = \sqrt{(1.0780771632\text{E} - 05)^2}$ =1.0780771632E-05

Vertical Standard Error $_E = \sqrt{(7.3577460175\text{E} - 06)^2}$ =7.3577460175E-06

Vertical Standard Error $_F = \sqrt{(6.7420171521\text{E} - 06)^2}$ =6.7420171521E-06

Table 6.10: Horizontal and Vertical Standard Error in 15 Minutes.

Horizontal and Vertical Standard Error in 15 Minutes.			
Point	**Name**	**Horizontal**	**Vertical**
1	**C**	7.97827E-06	4.8617445492E-06
2	**D**	1.1967E-05	1.0780771632E-05
3	**E**	1.07217E-05	7.3577460175E-06
4	**F**	1.07237E-05	6.7420171521E-06

After applying Least Squares(see appendix A) Computed Reference Variance at time 30 Minutes $V^T WV = 4.0311E\text{-}08$.

$S_0 = \sqrt{\frac{V^T WV}{r}}$, (r) Degrees of Freedom =*12* , S_0 = 5.7959E-05

Computed Standard Deviation 30 Minutes in Station A.

$S_{XC} = S_0 \sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{1.0023469186E-06}$ =5.802700567E-08

$S_{YC} = S_0 \sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{9.4356207063E-07}$ = 5.629973726E-08

$S_{ZC} = S_0 \sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{1.82693749135E-07}$ =2.477324940E-08

$S_{XD} = S_0 \sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{2.9725882821E-06}$ = 9.992830283E-08

$S_{YD} = S_0 \sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{2.3067734271E-06}$ =8.802856063E-08

$S_{ZD} = S_0 \sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{3.08417931245E-06}$ = 1.017866780E-07

$S_{XE} = S_0 \sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{1.8345678751E-06}$ =7.850331981E-08

$S_{YE} = S_0 \sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{2.4567734922E-07}$ =2.872788737E-08

$S_{ZE} = S_0 \sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{2.03961741556E-06}$ =8.277429424E-08

$S_{XF}= S_0\sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{2.3452169230E-06}$ =8.875904901E-08

$S_{YF}= S_0\sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{1.0236131489E-06}$ = 5.863933800E-08

$S_{ZF}= S_0\sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{2.68300411904E-06}$ =9.493619042E-08

Table 6.11: Standard Deviation 30 Minutes in Station A.

Standard Deviation 30 Minutes in Station A.				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	5.62997373E-08	5.8027E-08	2.47732493991E-08
2	**D**	8.80285606E-08	9.9928302829E-08	1.01786678005E-07
3	**E**	2.87278874E-08	7.8503319807E-08	8.27742942426E-08
4	**F**	5.86393380E-08	8.8759049007E-08	9.49361904153E-08

Computed Standard Deviation 30 Minutes in Station B.

$S_{XC} = S_0\sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{5.6651766452E-06}$ =1.379519547665E-07

$S_{YC} = S_0\sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{5.1478239513E-06}$ = 1.315021864631E-07

$S_{ZC} = S_0\sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{1.28457663401E-07}$ =2.077309177275E-08

$S_{XD} = S_0\sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{8.4500523451E-06}$ = 1.684809607393E-07

$S_{YD}= S_0\sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{1.0236885548E-05}$ = 1.854406985466E-07

$S_{ZD}= S_0\sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{1.93281672882E-06}$ = 8.057799720529E-08

$S_{XE} = S_0\sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{6.0402213709E-06}$ =1.424451144542E-07

$S_{YE}= S_0\sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{6.6639245658E-06}$ =1.496187998840E-07

$S_{ZE}= S_0\sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{6.34248567138E-06}$ =1.459657214301E-07

$S_{XF}= S_0\sqrt{Q_{Xi\,Xi}}$ = 5.7959E-05 $\sqrt{8.1471391324E-06}$ =1.654335911251E-07

$S_{YF}= S_0\sqrt{Q_{Yi\,Yi}}$ = 5.7959E-05 $\sqrt{8.3062945274E-06}$ = 1.670416587380E-07

$S_{ZF}= S_0\sqrt{Q_{Zi\,Zi}}$ =5.7959E-05 $\sqrt{4.98585534905E-06}$ =1.294168926300E-07

Table 6.12: Standard Deviation 30 Minutes in Station B.

Standard Deviation 30 Minutes in Station B				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	1.31502186E-07	1.3795195477E-07	2.07730917728E-08
2	**D**	1.85440699E-07	1.6848096074E-07	8.05779972053E-08
3	**E**	1.49618800E-07	1.4244511445E-07	1.45965721430E-07
4	**F**	1.67041659E-07	1.6543359113E-07	1.29416892630E-07

Computed Average standard Deviation 30 Minutes in Station A and B.

Average standard Deviation at point = $\frac{\text{standard Deviation A} + \text{standard Deviation B}}{2}$

$S_{XC}=\frac{5.802700567E-08 + 1.379519547665E-07}{2}$ =9.7989480219E-08

$S_{YC}=\frac{5.629973726E-08 + 1.315021864631E-07}{2}$ =9.3900961863E-08

$S_{ZC}=\frac{2.477324940E-08 + 2.077309177275E-08}{2}$ =2.2773170586E-08

$S_{XD}=\frac{9.992830283E-08 + 1.684809607393E-07}{2}$ =1.3420463178E-07

$S_{YD}=\frac{8.802856063E-08 + 1.854406985466E-07}{2}$ =1.3673462959E-07

$S_{ZD}=\frac{1.017866780E-07+8.057799720529E-08}{2}$ =9.1182337605E-08

$S_{XE}=\frac{7.850331981E-08+1.424451144542E-07}{2}$ =1.1047421713E-07

$S_{YE}=\frac{2.872788737E-08+1.496187998840E-07}{2}$ =8.9173343629E-08

$S_{ZE}=\frac{8.277429424E-08+1.459657214301E-07}{2}$ =1.1437000784E-07

$S_{XF}=\frac{8.875904901E-08+1.654335911251E-07}{2}$ =1.2709632007E-07

$S_{YF}=\frac{5.863933800E-08+1.670416587380E-07}{2}$ =1.1284049837E-07

$S_{ZF}=\frac{9.493619042E-08+1.294168926300E-07}{2}$ =1.1217654152E-07

Table 6.13: Average Standard Deviation 30 Minutes.

Standard Average Deviation 30 Minutes.				
Point	**Name**	**Northing(Y)**	**Easting(X)**	**Elevation(Z)**
1	**C**	9.3900961863E-08	9.7989480219E-08	2.2773170586E-08
2	**D**	1.3673462959E-07	1.3420463178E-07	9.1182337605E-08
3	**E**	8.9173343629E-08	1.1047421713E-07	1.1437000784E-07
4	**F**	1.1284049837E-07	1.2709632007E-07	1.1217654152E-07

Computed Horizontal Standard Error the point C, D, E and F at the time 30 Minutes.

Horizontal Standard Error = $\sqrt{(S_X)^2+(S_Y)^2}$

Horizontal Standard $_C$ = $\sqrt{(9.7989480219E-08)^2+(9.3900961863E-08)^2}$ = 1.3571782E-7

Horizontal Standard $_D = \sqrt{(1.3420463178E-07)^2 + (1.3673462959E-07)^2}$ = 1.9159134E-7

Horizontal Standard $_E = \sqrt{(1.1047421713E-07)^2 + (8.9173343629E-08)^2}$ = 1.4197337E-7

Horizontal Standard $_F = \sqrt{(1.2709632007E-07)^2 + (1.1284049837E-07)^2}$ = 1.6996015E-7

Computed Vertical Standard Error the point C,D,E and F at the time 30 Minutes.

Vertical Standard Error $= \sqrt{(S_Z)^2}$

Vertical Standard Error $_C = \sqrt{(2.2773170586E-08)^2}$ =2.2773170586E-08

Vertical Standard Error $_D = \sqrt{(9.1182337605E-08)^2}$ =9.1182337605E-08

Vertical Standard Error $_E = \sqrt{(1.1437000784E-07)^2}$ =1.1437000784E-07

Vertical Standard Error $_F = \sqrt{(1.1217654152E-07)^2}$ =1.1217654152E-07

Table 6.14: Horizontal and Vertical Standard Error in 30 Minutes.

Horizontal and Vertical Standard Error in 30 Minutes.			
Point	**Name**	**Horizontal**	**Vertical**
1	**C**	1.35718E-07	2.2773170586E-08
2	**D**	1.91591E-07	9.1182337605E-08
3	**E**	1.41973E-07	1.1437000784E-07
4	**F**	1.6996E-07	1.1217654152E-07

6.2 Accuracy in Static.

This section shows data for the Standard deviations at each point and each method. The data for the deviations are presented in tables and in diagrams.

6.2.1 Curves of standard error, horizontal

The horizontal standard errors for Static observations are in this section presented in diagrams for each point separately where x-axis denotes different measuring times:

1 = 8 minutes, 2 = 15 minutes, 3 = 30 minutes and y-axis denotes standard error.

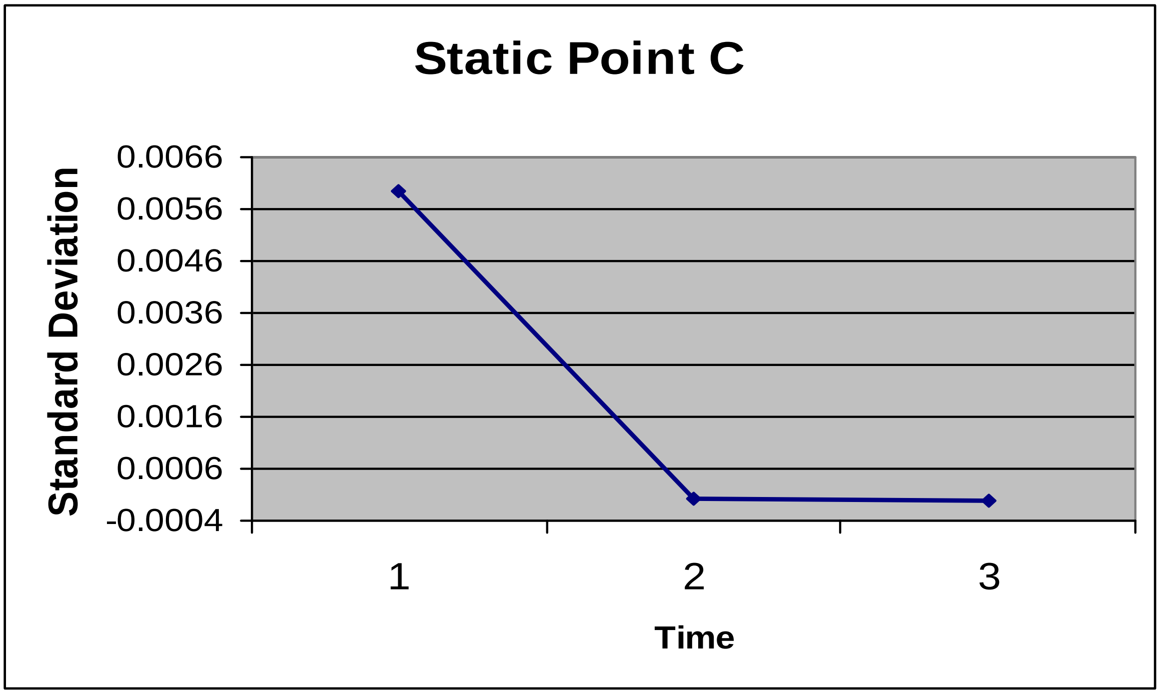


Figure 6.1: Standard error (horizontal) for Static Point C.

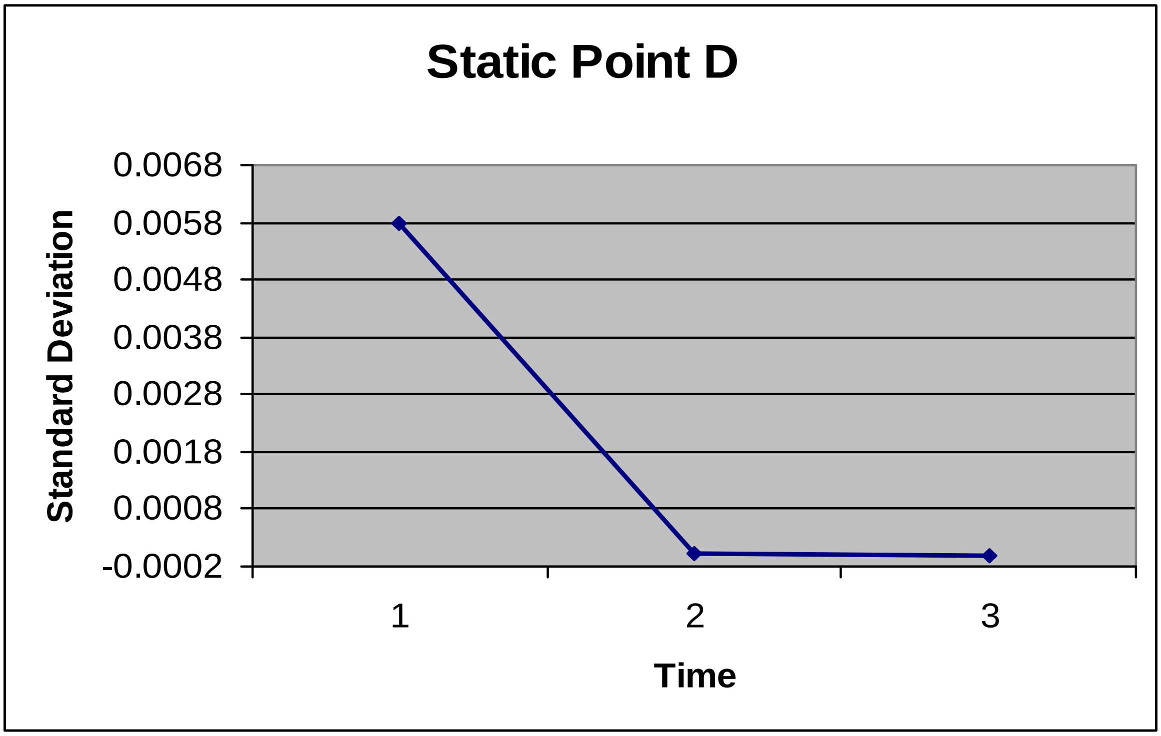
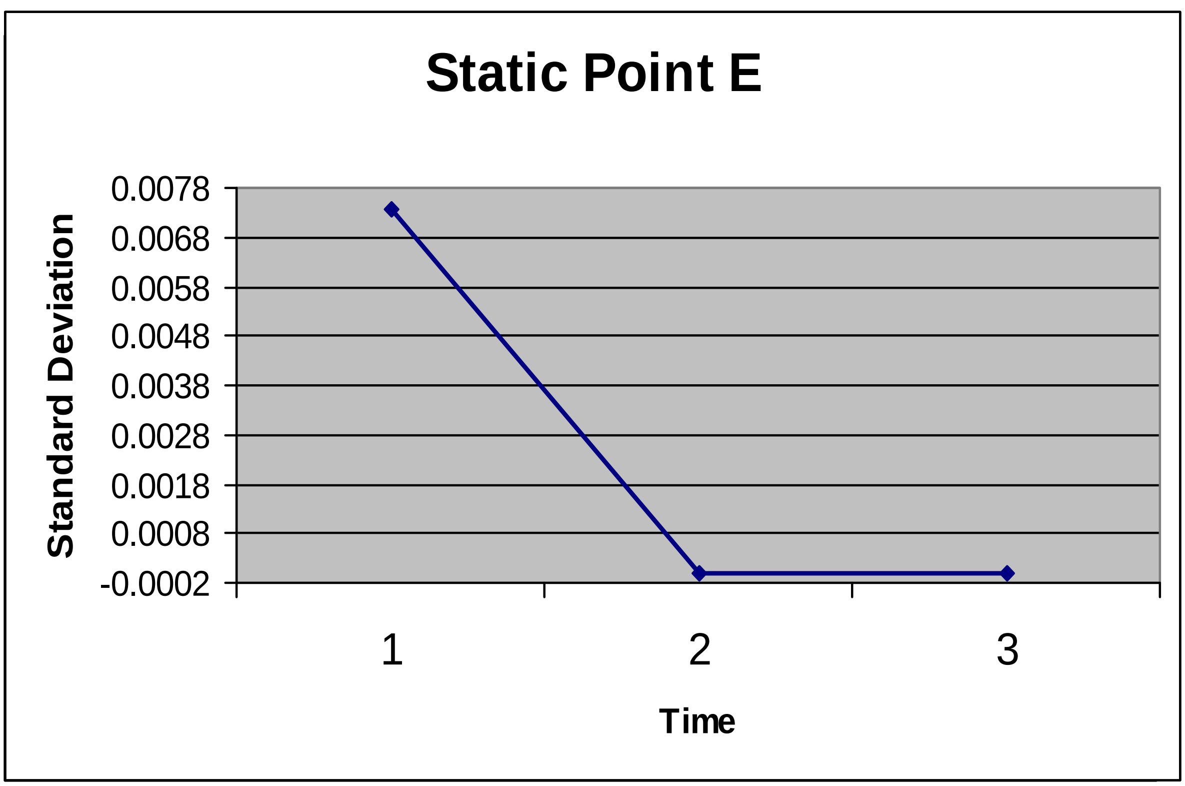


Figure 6.2: standard error (horizontal) for Static Point D.



6.2.2 Curves of standard error, vertical

The vertical standard errors for Static in this section are presented in diagrams for each point separately where x-axis denotes different measuring times: 1 = 8 minutes, 2 = 15 minutes, 3 = 30 minutes and y-axis denotes standard error.

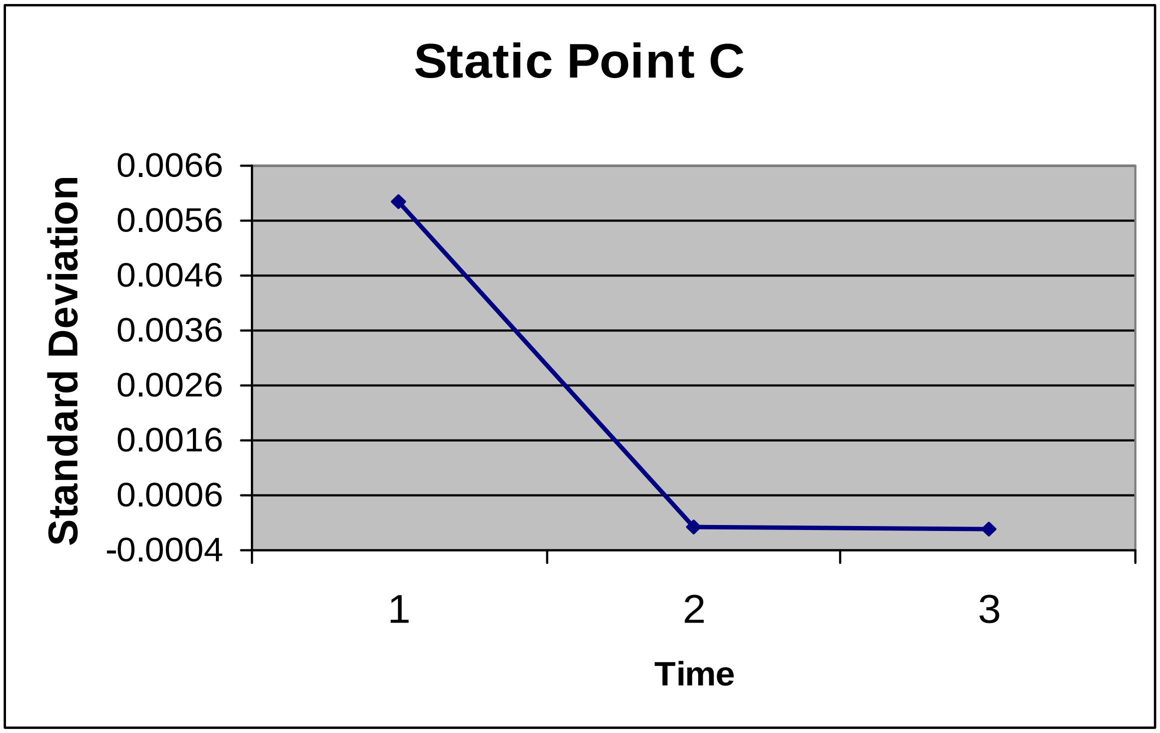
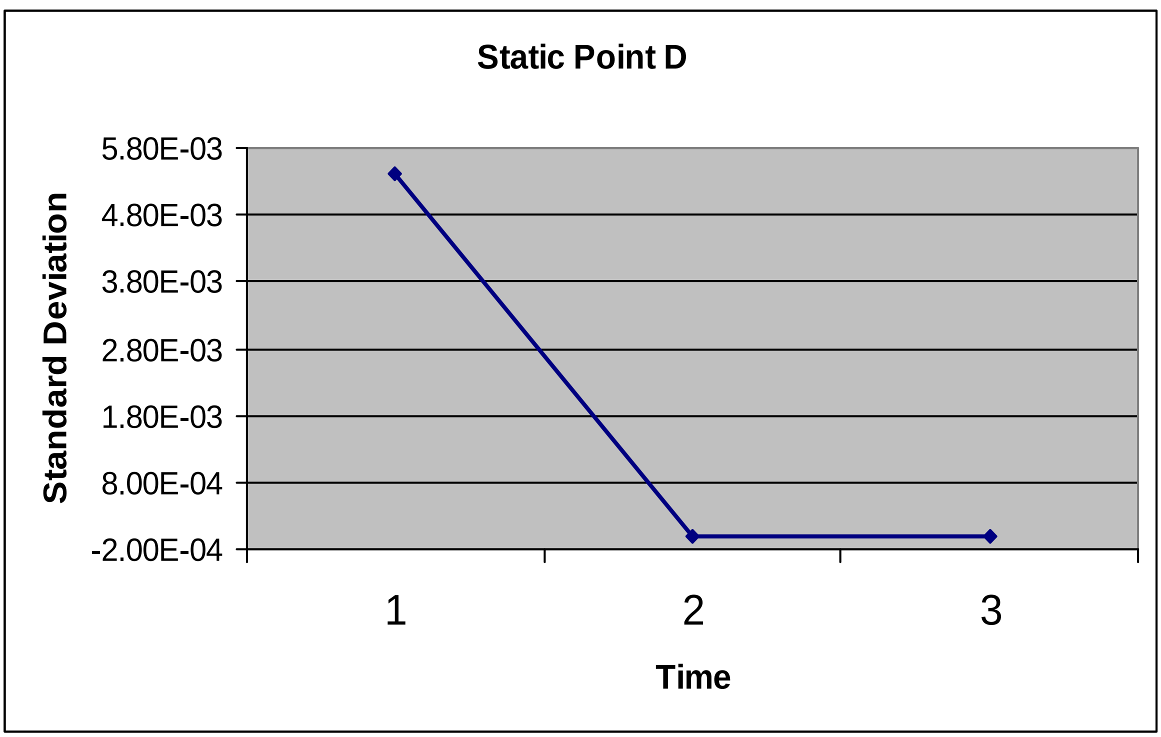


Figure 6.5: Standard error (vertical) for Static Point C



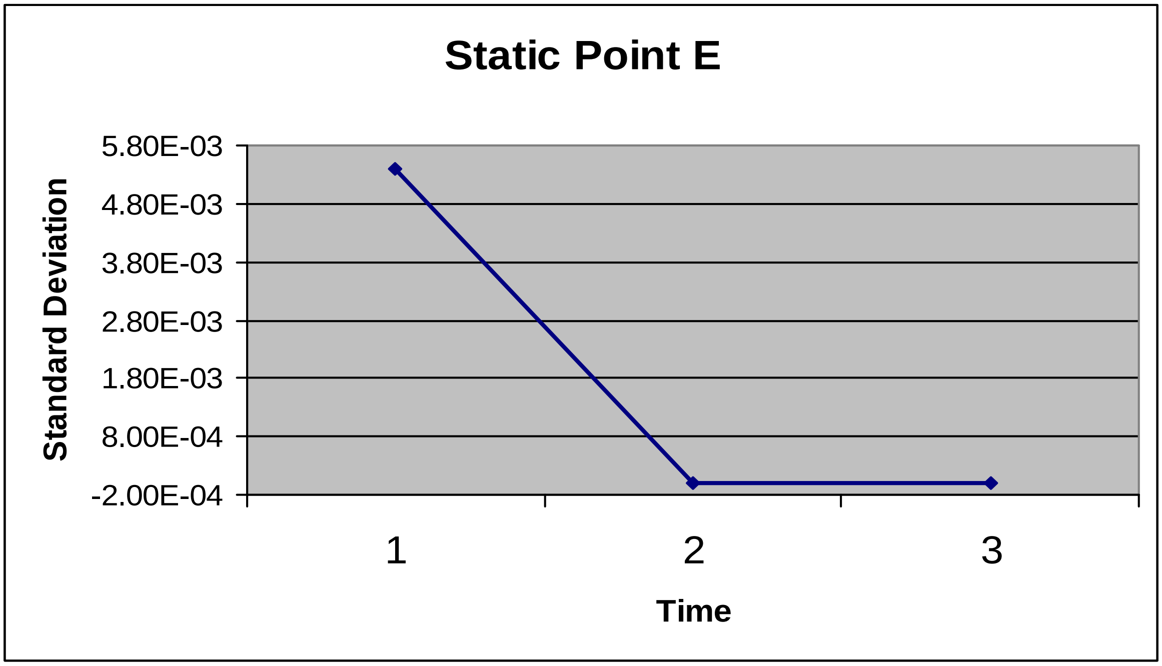


Figure 6.7: Standard error (vertical) for Static Point E

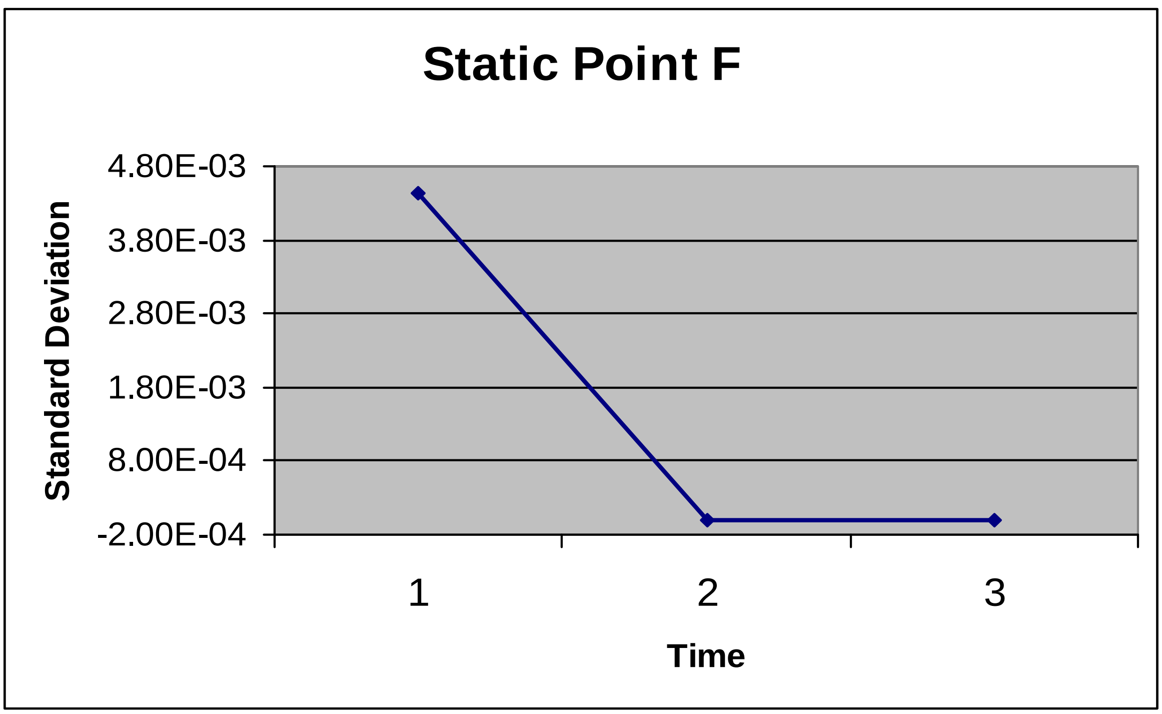


Figure 6.8: Standard error (vertical) for Static Point F

6.3 Accuracy in RTK.

All the calculations, made in this section are done with the TGO- software (Trimble Geomatics Office).

Table 6.15: Horizontal and Vertical Standard Error in 10 Seconds.

Horizontal and Vertical Standard Error in 10 Seconds.			
Point	**Name**	**Horizontal**	**Vertical**
1	**C**	0.408	0.013
2	**D**	0.007	0.0101
3	**E**	0.367	0.011
4	**F**	0.0071	0.012

Table 6.17: Horizontal and Vertical Standard Error in 1 Minutes

Horizontal and Vertical Standard Error in 1 Minutes			
Point	**Name**	**Horizontal**	**Vertical**
1	**C**	0.007	0.016
2	**D**	0.006	0.016
3	**E**	0.005	0.013
4	**F**	0.0073	0.018

Table 6.17: Horizontal and Vertical Standard Error in 5 Minutes.

Horizontal and Vertical Standard Error in 5 Minutes.			
Point	**Name**	**Horizontal**	**Vertical**
1	**C**	0.408	0.535
2	**D**	0.007	0.01
3	**E**	0.367	0.577
4	**F**	0.0065	0.014

6.3.1 Curves of standard error, horizontal

The horizontal standard errors for RTK in this section are presented in diagrams for each point separately where x-axis denotes different measuring times: 1 = 10 seconds, 2 = 1 minute, and value 3= 5 minutes and y-axis denotes standard error.

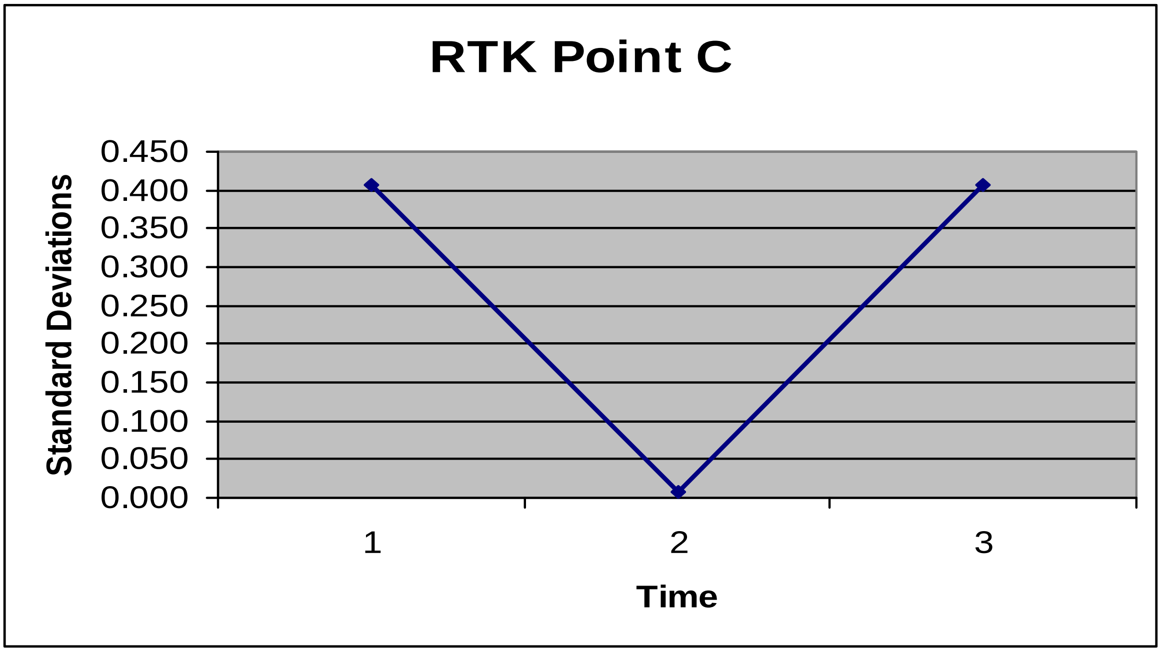


Figure 6.9: Standard error (horizontal) for RTK Point C

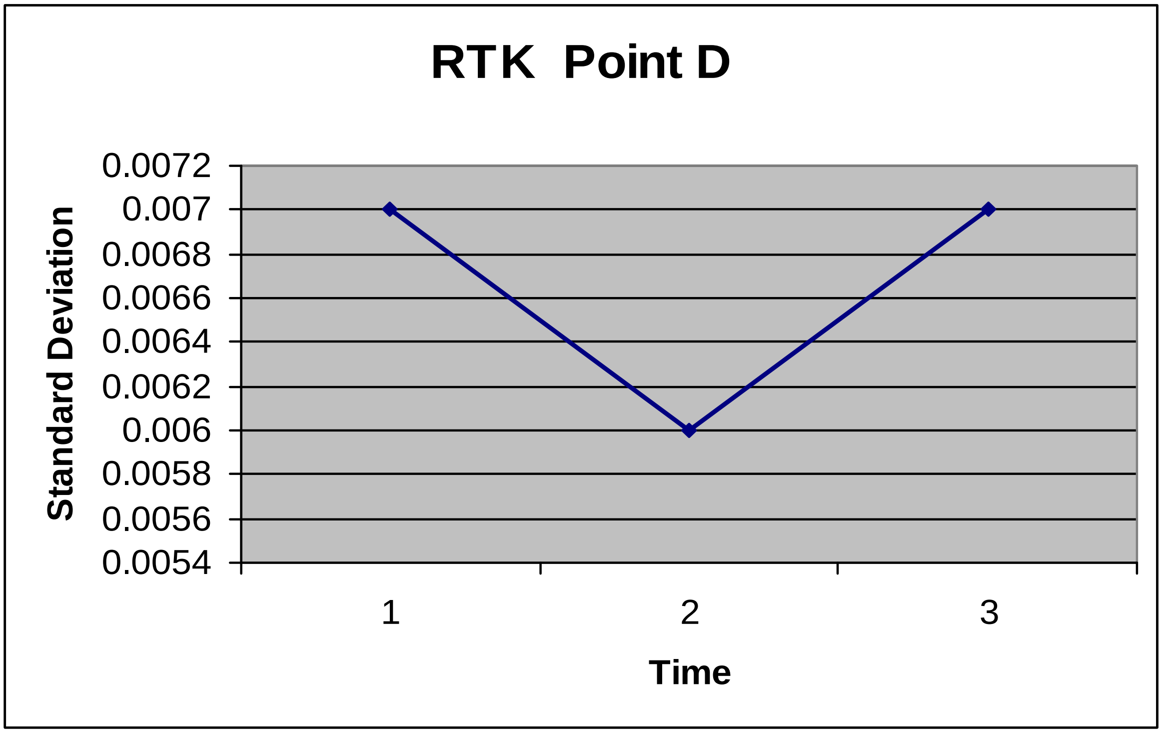


Figure 6.10: Standard error (horizontal) for RTK Point D

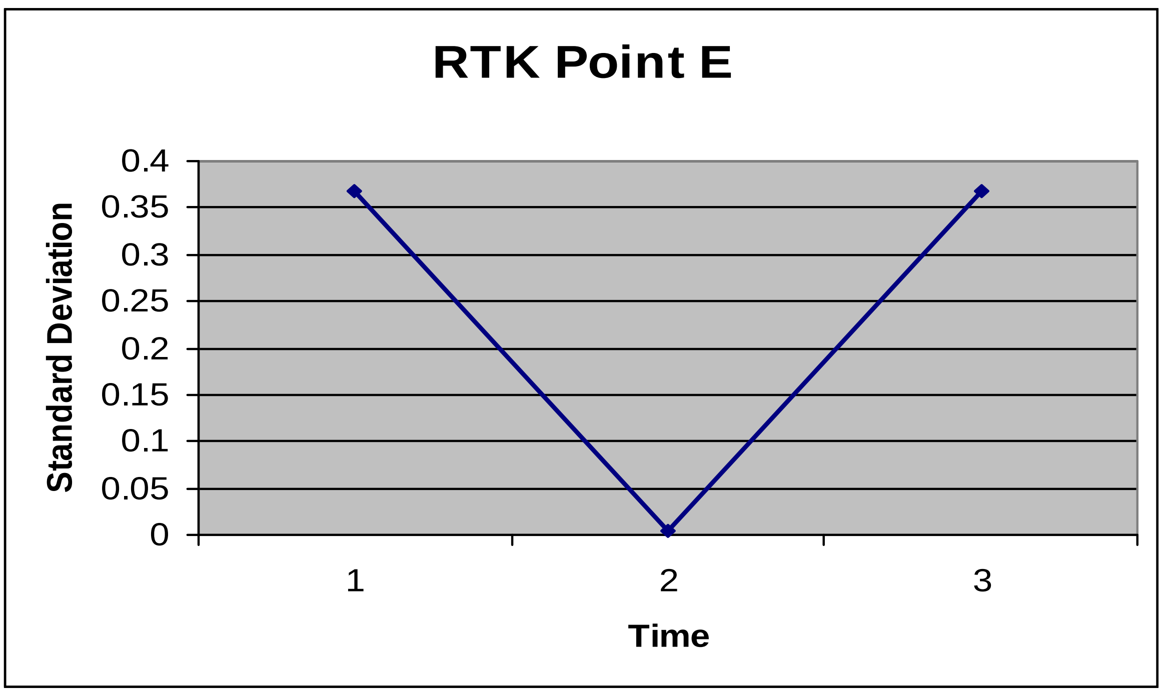


Figure 6.11: Standard error (horizontal) for RTK Point E

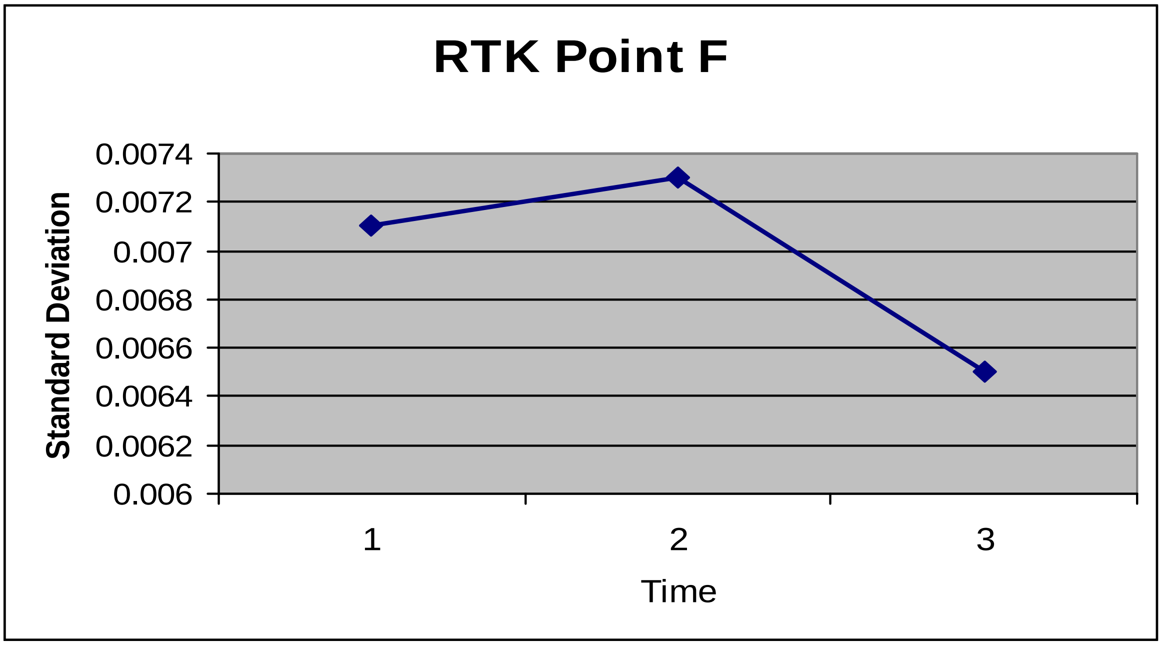


Figure 6.12: Standard error (horizontal) for RTK Point F

6.3.2 Curves of standard error, vertical

The vertical standard errors for RTK in this section are presented in diagrams for each point separately where x-axis denotes different measuring times: 1 = 10 seconds, 2 = 1 minute, and value 3 = 5 minutes and y-axis denotes standard error.

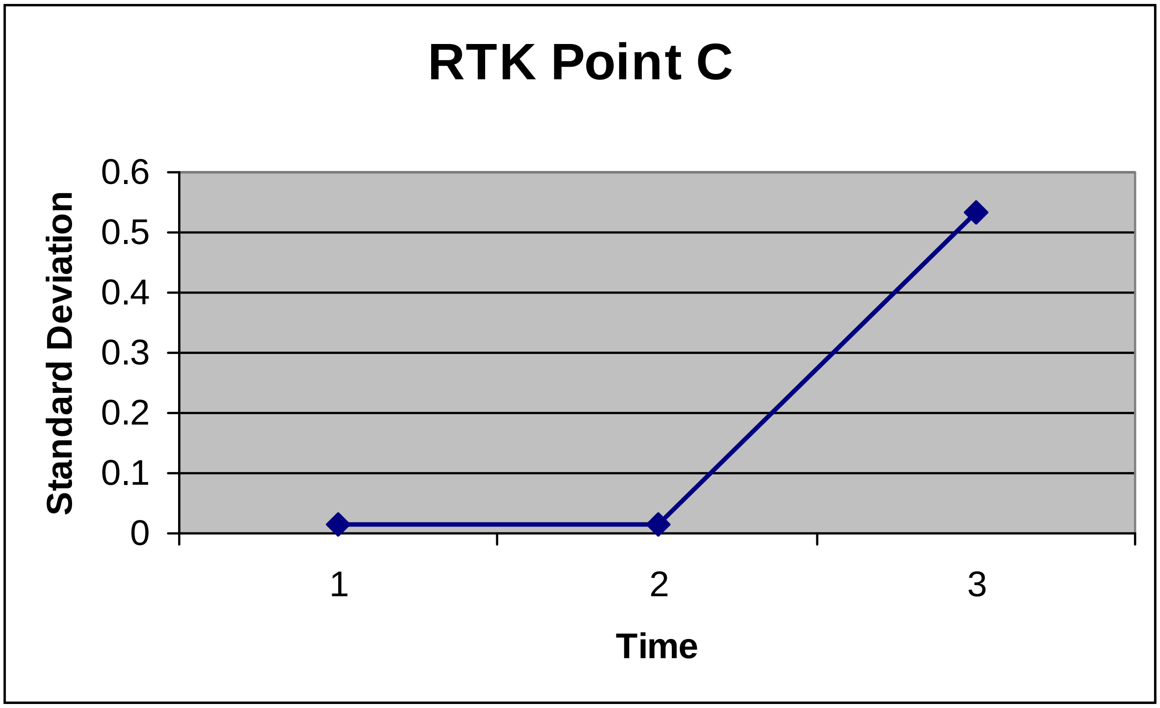


Figure 6.13: Standard error (vertical) for RTK Point C

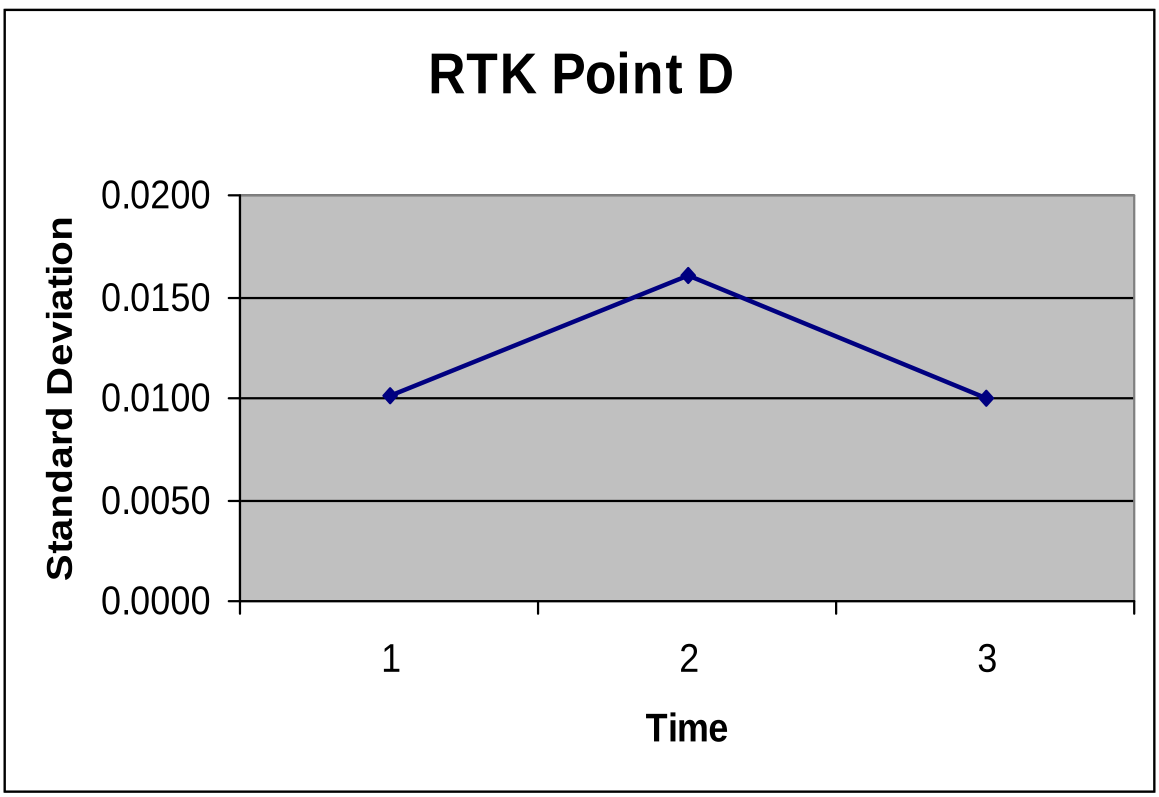


Figure 6.14: standard error (vertical) for RTK Point D

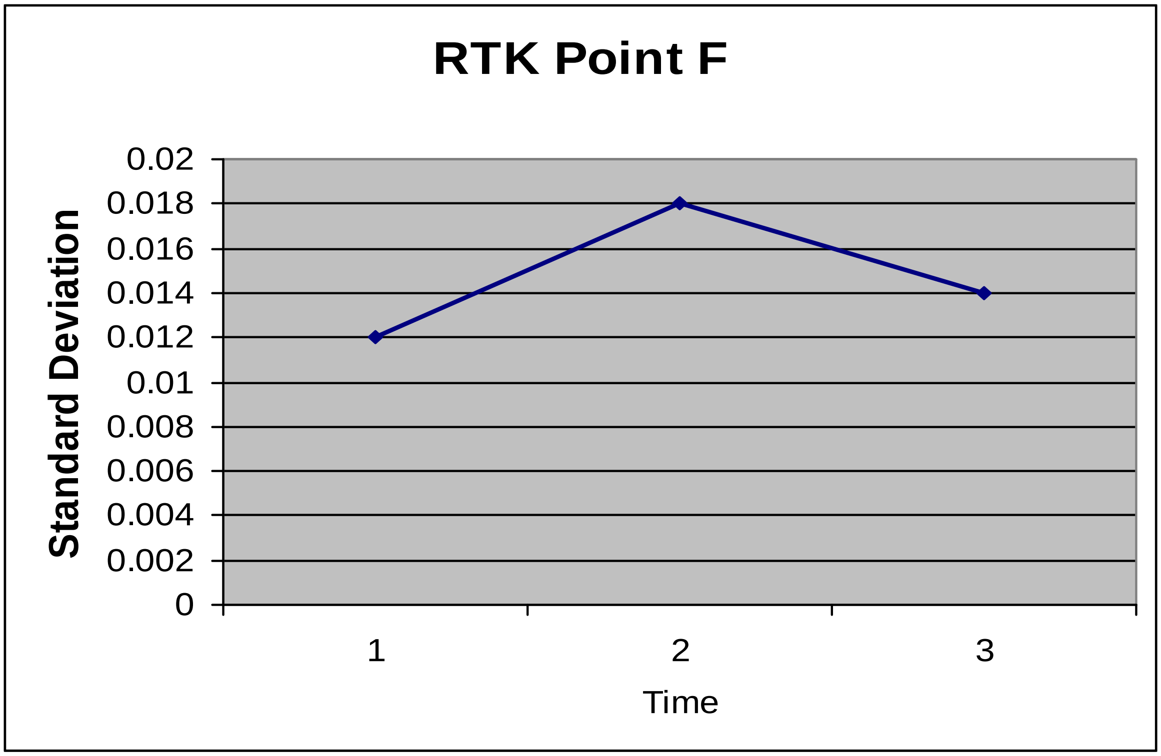


Figure 6.15: Standard error (vertical) for RTK Point F

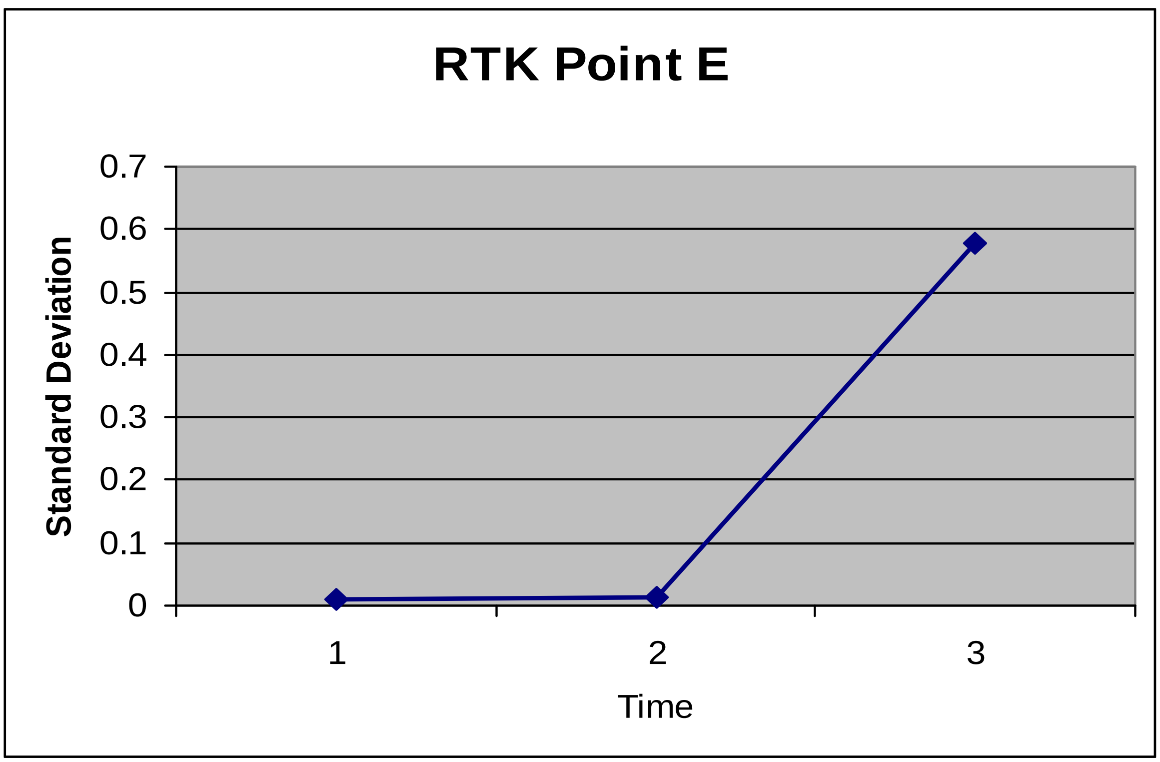


Figure 6.16: Standard error (vertical) for RTK Point E

6.4 Mean Standard Deviation

Table 6.12: is showing the Mean Standard Deviation Horizontal, Vertical and Mean 3D Standard Deviation

6.4.1. Mean Standard Deviation static

6.4.1.1 Mean Horizontal:

Mean Horizontal 8 Minute

$$= \frac{(0.005937057) + (0.005769237) + (0.007385911) + (0.006076162)}{4} = 6.29209175 \times 10^{-3}$$ Mean Horizonta1 15 Minute*:*

$$= \frac{(7.97827 \times 10^{-6}) + (1.1967 \times 10^{-5}) + (1.07217 \times 10^{-5}) + (1.07237 \times 10^{-5})}{4} = 1.03476675 \times 10^{-5}$$ Mean Horizontal 30Minute:

$$= \frac{[(1.35718) + (1.91591) + (1.41973) + (1.6996)] \times 10^{-7}}{4} = 1.598105 \times 10^{-7}$$

Table 6.18: Mean Static Standard Deviation (Horizontal).

Name	Mean Static Standard Deviation Horizontal		
	8 Minute	**15 Minute**	**30Minute**
C	0.005937057	7.98E-06	1.36E-07
D	0.005769237	1.20E-05	1.92E-07
E	0.007385911	1.07E-05	1.42E-07
F	0.006076162	1.07E-05	1.70E-07
Mean	6.29E-03	1.03E-05	1.598105E-07

6.4.1.2 Mean Vertical:

Mean Vertical 8 Minute :

$$= \frac{[(4.3564382173) + (4.0254909579) + (5.4016402256) + (4.4484663360)] \times 10^{-3}}{4} = 4.5580689 \times 10^{-3}$$

Mean Vertical 15 Minute:

$$= \frac{[(4.8617445492) + (10.780771632) + (7.3577460175) + (6.7420171521)] \times 10^{-6}}{4} = 7.4355698 \times 10^{-6}$$

Mean Vertical 30Minute:

$$= \frac{[(2.2773170586) + (9.1182337605) + (11.437000784) + (11.217654152)] \times 10^{-8}}{4} = 8.5125514 \times 10^{-8}$$

Table 6.19: Mean Static Standard Deviation (Vertical).

Name	**Mean Static Standard Deviation Vertical**		
	8 Minute	**15 Minute**	**30Minute**
C	4.36E-03	4.86E-06	2.28E-08
D	4.03E-03	1.08E-05	9.12E-08
E	5.40E-03	7.36E-06	1.14E-07
F	4.45E-03	6.74E-06	1.12E-07
Mean	4.56E-03	7.44E-06	8.51E-08

6.4.1.3 Total precision (3D).

Table 6.20: is showing the total precision 3D standard deviations divided separately for the different time epochs.

3D = $\sqrt{(¢_X)^2 + (¢_Y)^2 + (¢_Z)^2}$

For 8 Minutes:

$$3D_C = \sqrt{(0.005937057)^2 + (4.3564382173 \times 10^{-3})^2} = 7.3639110 \times 10^{-3}$$

$$3D_D = \sqrt{(0.005769237)^2 + (4.0254909579 \times 10^{-3})^2} = 7.0348186 \times 10^{-3}$$

$$3D_E = \sqrt{(0.007385911)^2 + (5.4016402256 \times 10^{-3})^2} = 9.15037690 \times 10^{-3}$$

$$3D_F = \sqrt{(0.006076162)^2 + (4.4484663360 \times 10^{-3})^2} = 7.5305110 \times 10^{-3}$$

Mean 3D:

$$= \frac{[(7.363911) + (7.0348186) + (9.1503769) + (7.530511)] \times 10^{-3}}{4} = 7.769904 \times 10^{-3}$$

For 15 Minutes:

$$3D_C = \sqrt{(7.97827 \times 10^{-6})^2 + (4.8617445492 \times 10^{-6})^2} = 9.342877 \times 10^{-6}$$

$$3D_D = \sqrt{(1.1967 \times 10^{-5})^2 + (1.0780771632 \times 10^{-5})^2} = 1.6106958 \times 10^{-5}$$

$$3D_E = \sqrt{(1.07217 \times 10^{-5})^2 + (7.3577460175 \times 10^{-6})^2} = 1.300351019 \times 10^{-5}$$

$$3D_F = \sqrt{(1.07237 \times 10^{-5})^2 + (6.7420171521 \times 10^{-6})^2} = 1.2666986 \times 10^{-5}$$

Mean 3D:

$$= \frac{[(0.9342877) + (1.6106958) + (1.300351019) + (1.2666986)] \times 10^{-5}}{4} = 1.278008282 \times 10^{-5}$$

For 30 Minutes:

$$3D_C = \sqrt{(1.35718 \times 10^{-7})^2 + (2.2773170586 \times 10^{-8})^2} = 1.3761538 \times 10^{-7}$$

$$3D_D = \sqrt{(1.91591 \times 10^{-7})^2 + (9.1182337605 \times 10^{-8})^2} = 2.121823036 \times 10^{-7}$$

$$3D_E = \sqrt{(1.41973 \times 10^{-7})^2 + (1.1437000784 \times 10^{-7})^2} = 1.82309712 \times 10^{-7}$$

$$3D_F = \sqrt{(1.6996 \times 10^{-7})^2 + (1.1217654152 \times 10^{-7})^2} = 2.03641788 \times 10^{-7}$$

Mean 3D:

$$= \frac{[(1.3761538)+(2.121823036)+(1.82309712)+(2.03641788)]\times 10^{-7}}{4} = 1.83937296\times 10^{-7}$$

Table 6.20: Mean Static Standard Deviation (3D).

Name	Mean Static Standard Deviation 3D		
	8 Minute	**15 Minute**	**30Minute**
C	7.36E-03	9.34E-06	1.38E-07
D	7.03E-03	1.61E-05	2.12E-07
E	9.15E-03	1.30E-05	1.82E-07
F	7.53E-03	1.27E-05	2.04E-07
Mean	7.77E-03	1.28E-05	1.84E-07

Table 6.21: Mean Static Standard Deviation (H, V, 3D) different time epochs.

Time	**Mean Horizontal**	**Mean Vertical**	**Mean 3D**
8 Minute	6.29209E-03	4.56E-03	7.77E-03
15 Minute	1.03E-05	7.44E-06	1.28E-05
30 Minute	1.598E-07	8.513E-08	1.839E-07

6.4.2 Mean Standard Deviation RTK.

6.4.2.1 Mean Horizontal:

Mean Horizontal 10 Seconds $= \frac{(0.408)+(0.007)+(0.367)+(0.0071)}{4} = 0.197275$

$$\text{Mean Horizontal 1 Minute} = \frac{(0.007)+(0.006)+(0.005)+(0.0073)}{4} = 6.325\times 10^{-3}$$

$$\text{Mean Horizontal 5 Minute} = \frac{(0.408)+(0.007)+(0.367)+(0.0065)}{4} = 0.197125$$

Table 6.22: Mean RTK Standard Deviation (Horizontal).

Name	Mean RTK Standard Deviation Horizontal		
	10 Seconds	**1 Minute**	**5 Minute**
C	0.408	0.007	0.408
D	0.007	0.006	0.007
E	0.367	0.005	0.367
F	0.0071	0.0073	0.0065
Mean	0.197275	6.33E-03	0.197125

6.4.2.2 Mean Vertical:

$$\text{Mean Vertical 10 Seconds} = \frac{(0.013)+(0.0101)+(0.011)+(0.012)}{4} = 0.011525$$

$$\text{Mean Vertical 1 Minute} = \frac{(0.016)+(0.016)+(0.013)+(0.018)}{4} = 0.01575$$

$$\text{Mean Vertical 5 Minute} = \frac{(0.535)+(0.010)+(0.577)+(0.014)}{4} = 0.284$$

Table 6.23: Mean RTK Standard Deviation (Vertical).

Name	Mean RTK Standard Deviation Vertical		
	10 Seconds	**1 Minute**	**5 Minute**
C	0.013	0.016	0.535
D	0.0101	0.016	0.01
E	0.011	0.013	0.577
F	0.012	0.018	0..014
Mean	0.011525	0.01575	0.284

6.4.2.3 Total precision (3D).

Table 6.14 is showing the total precision 3D standard deviations divided separately for the different time epochs.

For 10 seconds:

$$3D_C = \sqrt{(0.408)^2 + (0.013)^2} = 0.408207$$

$$3D_D = \sqrt{(0.007)^2 + (0.0101)^2} = 0.0122886$$

$$3D_E = \sqrt{(0.367)^2 + (0.011)^2} = 0.367164$$

$$3D_F = \sqrt{(0.0071)^2 + (0.012)^2} = 0.013943$$

$$\text{Mean 3D} = \frac{(0.408207) + (0.0122886) + (0.367164) + (0.013943)}{4} = 0.20040065$$

For 1 minute:

$$3D_C = \sqrt{(0.007)^2 + (0.016)^2} = 0.017464$$

$$3D_D = \sqrt{(0.006)^2 + (0.016)^2} = 0.017088$$

$$3D_E = \sqrt{(0.005)^2 + (0.013)^2} = 0.0139283$$

$$3D_F = \sqrt{(0.0073)^2 + (0.018)^2} = 0.0194239$$

$$\text{Mean 3D} = \frac{(0.017464) + (0.017088) + (0.0139283) + (0.0194239)}{4} = 0.01697605$$

For 5 Minutes:

$$3D_C = \sqrt{(0.408)^2 + (0.535)^2} = 0.67282167$$

$$3D_D = \sqrt{(0.007)^2 + (0.010)^2} = 0.012206$$

$$3D_E = \sqrt{(0.367)^2 + (0.577)^2} = 0.683826$$

$$3D_F = \sqrt{(0.0065)^2 + (0.014)^2} = 0.0154353$$

$$\text{Mean 3D} = \frac{(0.67282167) + (0.012206) + (0.683826) + (0.0154353)}{4} = 0.3460722$$

Table 6.24: Mean RTK Standard Deviation (3D).

Name	Mean RTK Standard Deviation 3D		
	10 Seconds	**1 Minute**	**5 Minute**
C	0.408207	0.017464	0.67282
D	0.0122886	0.017088	0.012206
E	0.367164	0.0139283	0.683826
F	0.013943	0.0194239	0.0154353
Mean	0.20040065	0.01697605	0.3460722

Table 6.25 Mean RTK Standard Deviation (H, V, 3D) different time epochs.

Time	Mean Horizontal	Mean Vertical	Mean 3D
10 Seconds	1.9728E-01	1.1525E-02	2.0040E-01
1 Minute	6.3250E-03	1.5750E-02	1.6976E-02
5 Minute	1.9713E-01	2.8400E-01	3.4607E-01

6.5 Calculation best fit line.

Using the equation y = mx +b to compute the best fit mean 3D static and RTK.
Y is equal the standard Deviation, X is equal time, m is slope, and b is constant.
After substation in above equation.

Mean 3D static.

$$A = \begin{bmatrix} 8 & 1 \\ 15 & 1 \\ 30 & 1 \end{bmatrix}_{3\times 2} \qquad X = \begin{bmatrix} m \\ b \end{bmatrix}_{2\times 1} \qquad L = \begin{bmatrix} 7.769904 \times 10^{-3} \\ 1.2780082 \times 10^{-5} \\ 1.8393729 \times 10^{-7} \end{bmatrix}_{3\times 1}$$

X= $(A^T A)^{-1} A^T L$

M = -0.000297

B = 0.007848

Mean 3D RTK.

$$A = \begin{bmatrix} 0.1666 & 1 \\ 1 & 1 \\ 5 & 1 \end{bmatrix}_{3\times2} \quad X = \begin{bmatrix} m \\ b \end{bmatrix}_{2\times1} \quad L = \begin{bmatrix} 0.200040065 \\ 0.01697605 \\ 0.3460722 \end{bmatrix}_{3\times1}$$

$X = (A^T A)^{-1} A^T L$

M = 0.046625

B = 0.091975

6.6 Curves of static mean standard error.

The mean standard error, for Static observation in this section presented in diagrams for each point separately where x-axis denotes different measuring times 1=8minutes ,2=15 minutes,3=30 minutes and y-axis denotes standard error.

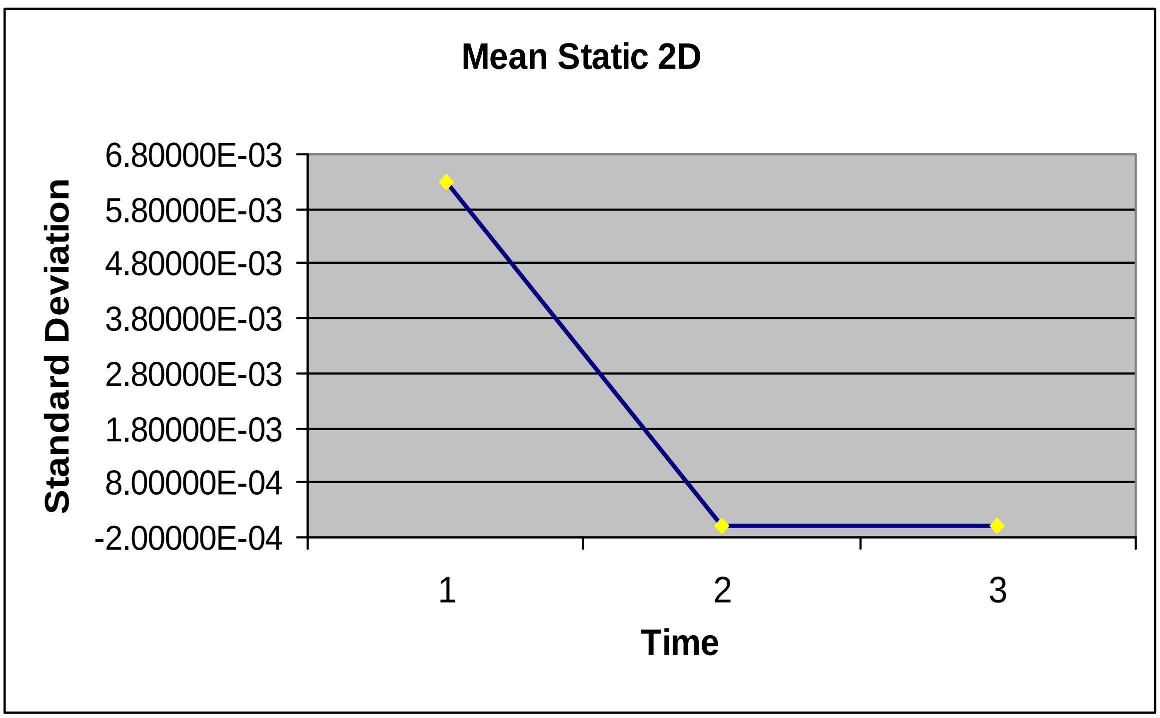


Figure 6.17: Mean standard error (2D) static

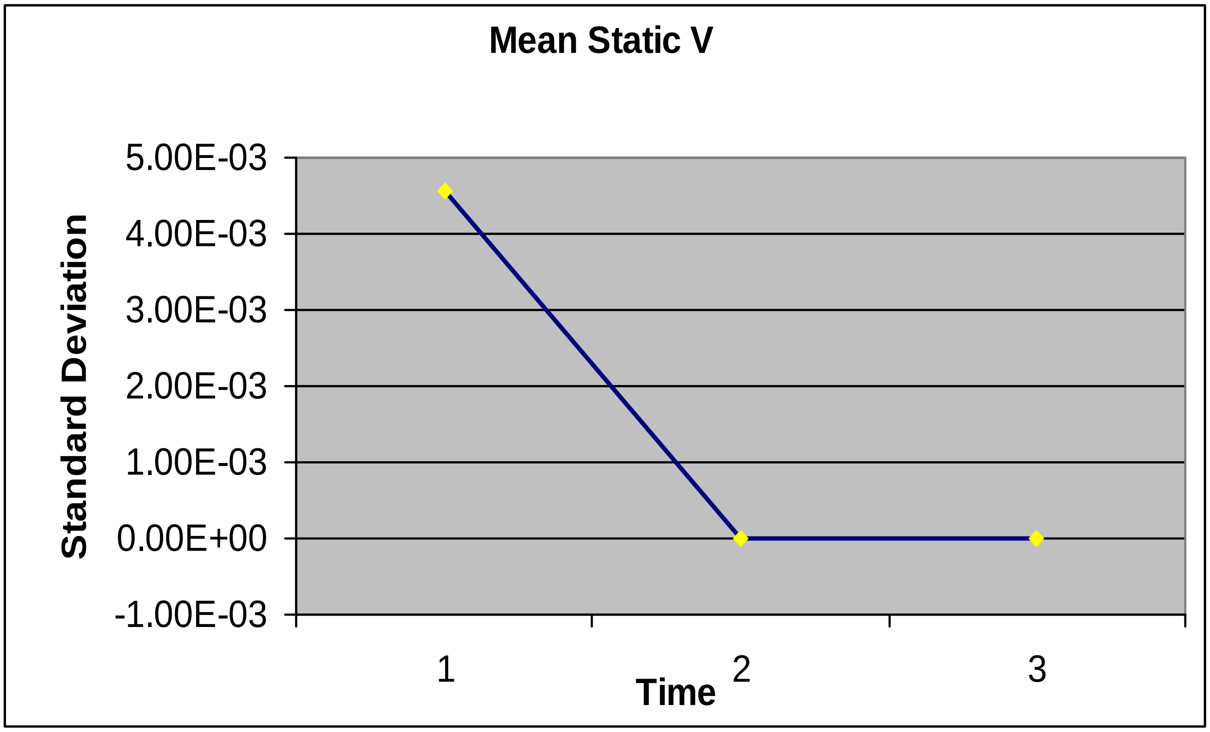


Figure 6.18: Mean standard error (V) static

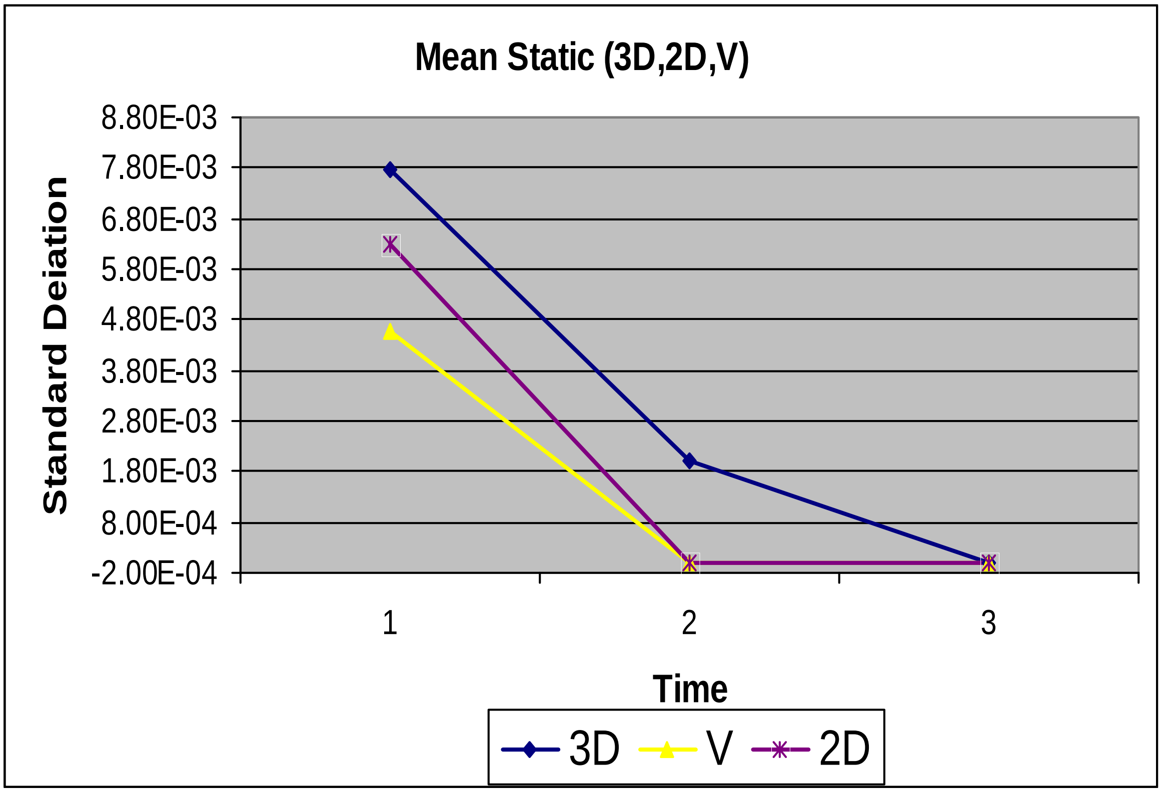


Figure 6.19: Mean standard error (3D, V, 2D) static

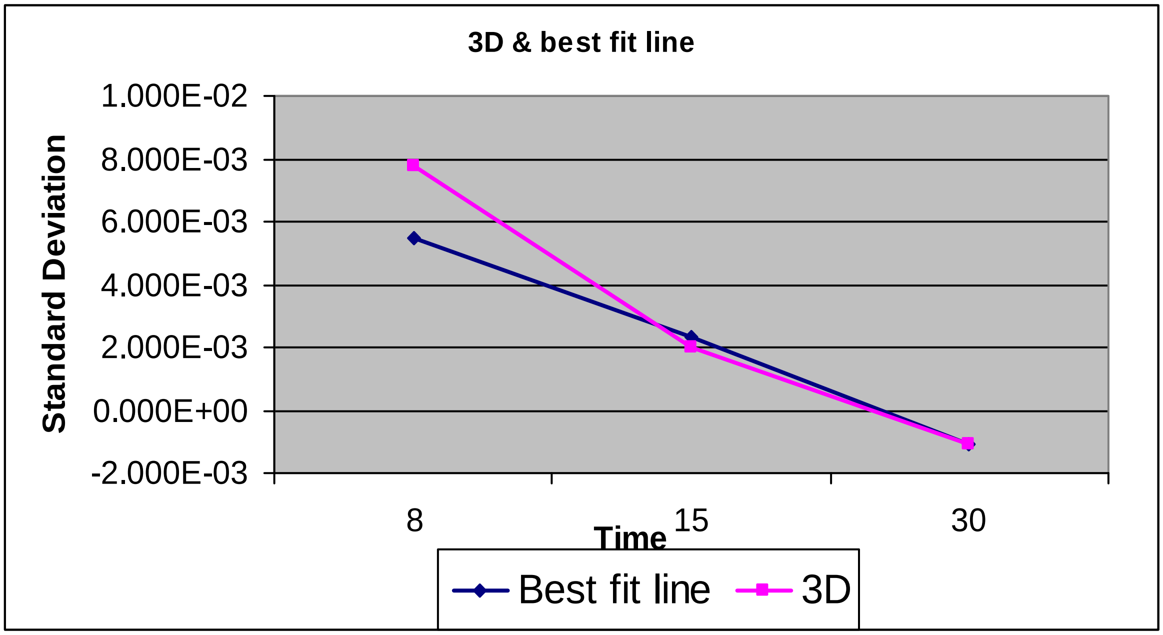


Figure 6.20: Mean standard error (3D, best fit) static

6.7 Curves of RTK mean standard error.

The mean standard error, for Static observation in this section presented in diagrams for each point separately where x-axis denotes different measuring times 1=10 second, 2 =1 minutes, 3 = 5 minutes and y-axis denotes standard error.

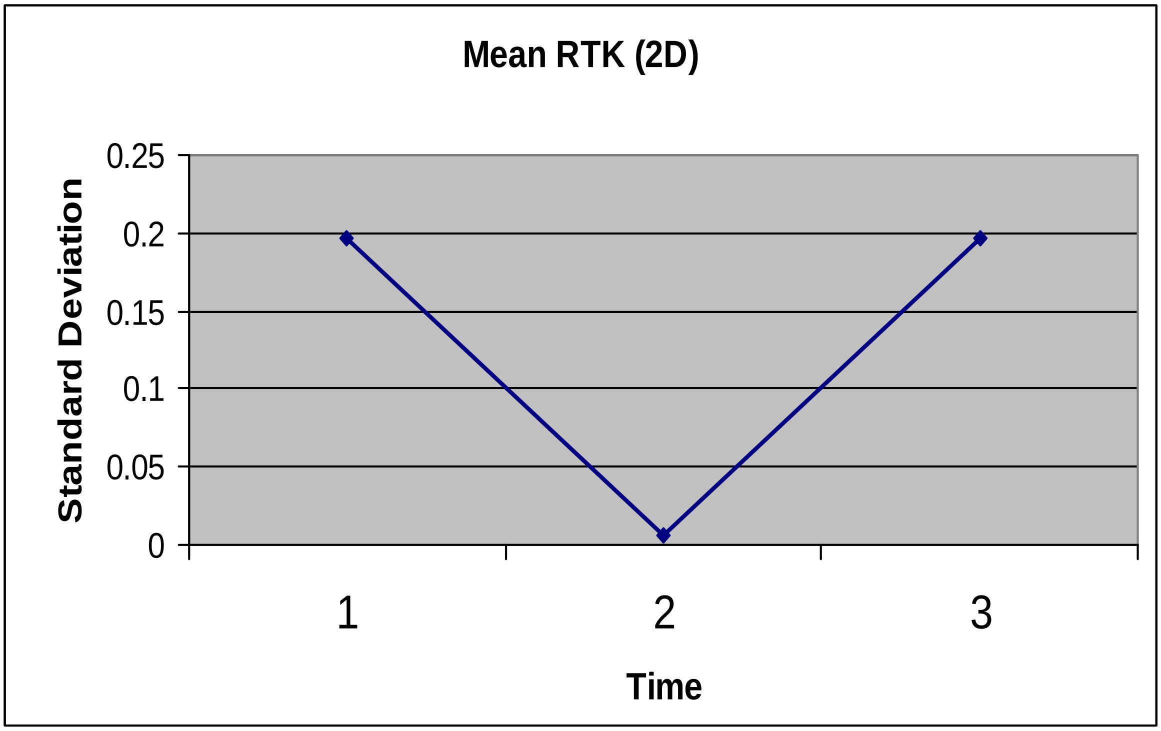


Figure 6.21: Mean standard error (2D) RTK

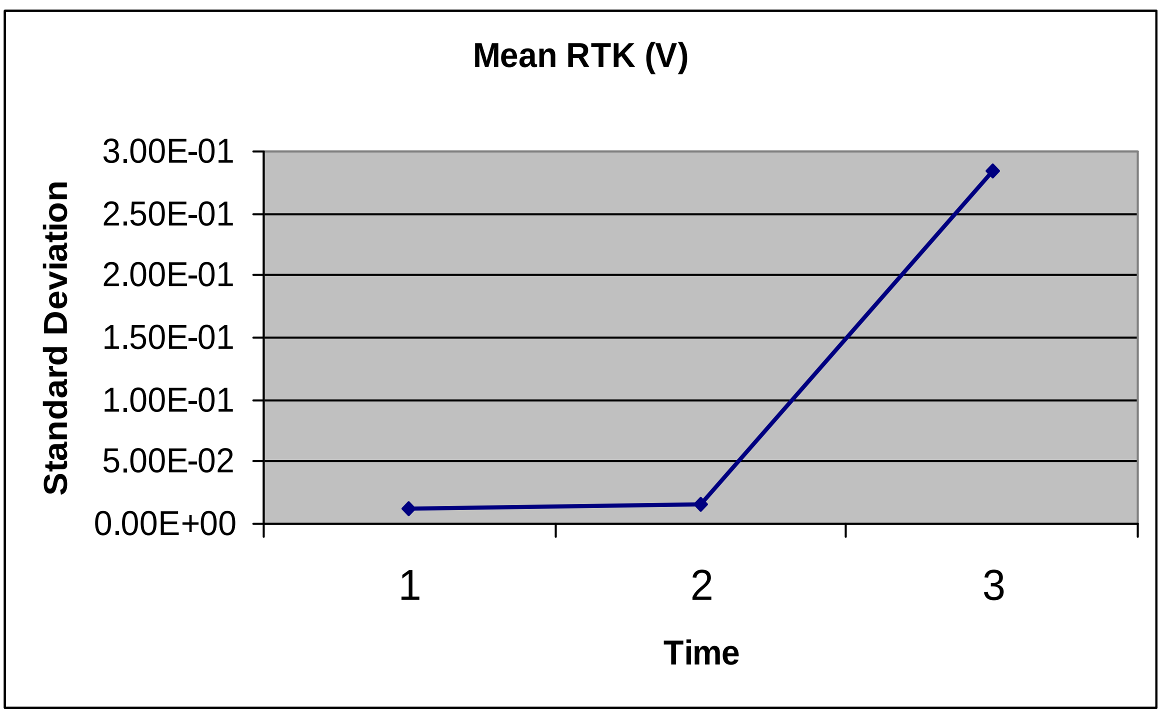


Figure 6.22: Mean standard error (V) RTK

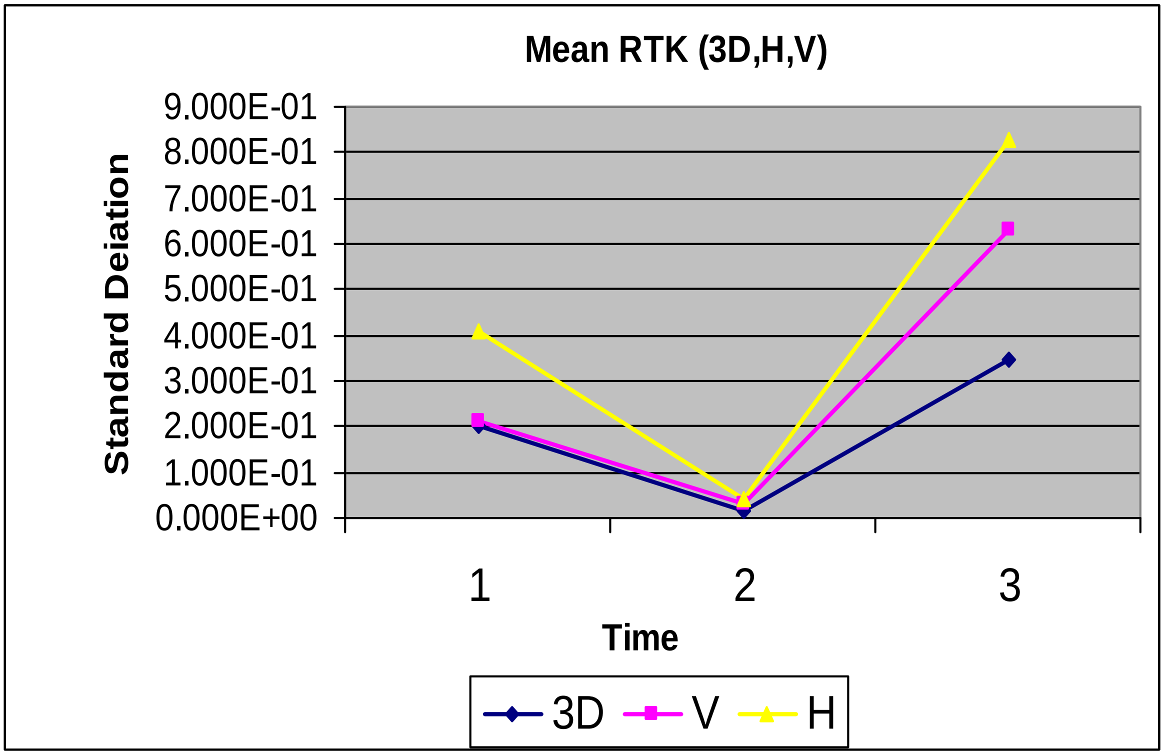


Figure 6.23: Mean standard error (3D,2D,V) RTK

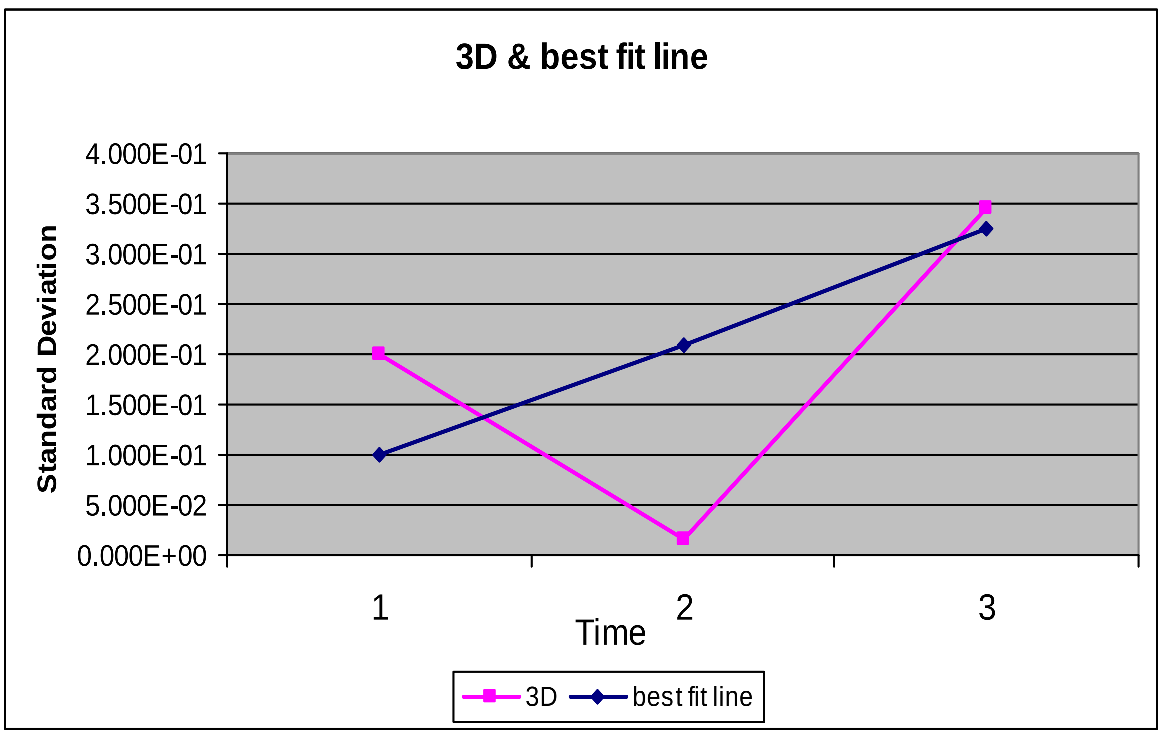


Figure 6.24: Mean standard error (3D,best fit) RTK.

6.8 General Formula Computed standard deviation

Standard deviation in 3D calculation are based on Standard deviation in X and Standard deviation in Y for given time period .
The standard deviation in horizontal frequency relation could be expressed by the formula:

$(S\text{-}D_{3D})_{\text{Static}} = B \times M^{T}$

Where $(S\text{-}D_{3D})$: standard deviation in total .
T : time(min).
B= 0.077789, M= 0.636172.

$(S\text{-}D_{3D})_{\text{RTK}} = B \times M^{T}$

Where $(S\text{-}D_{3D})$: standard deviation in total .
T : time(min).
B= 0.055192 , M= 1.371114

The values of "m" and "b" are determined using the data in the Table 6.21 in static and Table 6.25 in RTK , and using the help of Excel software .

6.9 Comparison.

The main task for this project is to compare the performance of two different GPS observation techniques regarding accuracy . The method used for these comparisons are RTK and static.
In static methods less accurate when measuring the points less than 15 minute(horizontal and vertical)accuracy is equal (6×10^{-3}) as showing figure and tables . Then the accuracy was better

when the time above the 15 minutes less than (6×10^{-4}) after 15 minute the accuracy increasing slowly.

Only human support was used so a big part of the error is related to the centering of the antenna over the point and measured antenna height.

Totally the 3D standard deviation of the static was few millimeter but RTK standard deviation in centimeter the reason for the accuracy depend on the initialization on GPS. The accuracy in vertical ,horizontal and total (3D) is different.

The accuracy didn't get notably better when observing for longer periods time in RTK . In static observing for longer period of time, as one hour, the measurement gets more reliable and the accuracy will get a bit better, but the question is if this bit is worth measuring for, for most cases we believe not.

After calculating the mean RTK accuracy was less when observing for long time. The different RTK accuracy refer to wait for satellites and not having connection with at least five satellites, the receiver could not determine phase ambiguities and that are needed for the initialization.

The slope of the line that best fitted the points in figure 6.20 was very near to -0.000297 it means that the accuracy is increasing -0.000297m/min and the slope in figure 6.23 was very near to 0.046625 it means that the accuracy is decreasing by 0.046625 m/min.

CHAPTER SEVEN

CONCLUSION AND RECOMMENDATION

This is the final Chapter in the project which includes the results “Conclusion " and recommendation that reached at the last of research.

7.1 Conclusions

7.2 Recommendations

CHAPTER SEVEN

CONCLUSION AND RECOMMENDATION

This chapter outlines a summary of the main conclusions made in this project. A more detailed discussion could be read in chapter five and six.

In this project is to compare the performance of two different GPS observation techniques regarding accuracy. The method used for these comparisons are RTK and static. The result of this study brought out many important conclusions. The main conclusions and recommendations drawn from the present study are summarized below.

7.1 Conclusions

1) Static better than RTK in accuracy.

2) The accuracy didn't notably improve when measuring with long time by RTK.

3) The accuracy in static notably improves when measuring with long time.

4) The accuracy in RTK changes at any time (increasing or decreasing).

7.2 Recommendations

1) Extend the study surveying methods using GPS.

2) Using normal single baseline RTK with GSM communication.

3) Using GPS in project surveying of road geometry with real time kinematics.

Reference

1. Teunissen peter J.G, GPS FOR GEODESY

2. Chamdra .A.M, HIGHER SURVEYING

3. Alfred Leick, GPS SATELLIITE SURVEYING

4. James M .Anderson, Edowarad M.Mikhail ,SURVEYING THEORE AND PRACTICE

5. Poul R.Wolf ,Charles D.Ghilani ,ADJUSTMENT COMPUTATIONS.

6. WWW.Trimble.com

7. www.navcen.uscg.gov

8. http://www.gmat.unsw.edu.au/snap/gps/gps_survey/

9. http://www.geoplane.com/support.html

10. http://www.geoplane.com/products.html

11. 5700/5800 GPS Receiver User Guide

12. Trimble Geomatics Office User Guide .