PALESTINE POLYTECHNIC UNIVERSITY

COLLEGE OF ENGINEERING AND TECHNOLOGY

CIVIL &ARCHITECTURAL ENGINEERING DEPARTMENT

SUREYING ENGINEERING

Graduation Project

Establishment of Palestine Polytechnic University (PPU) BaseLine and Geodetic Network

Prepared By:

Ahlam Alan **Ammar Al-Jabari**

Raghad Al-Sa'dah

Project Supervisor

Eng. FAYDI SHABANEH

HEBRON - PALESTINE

June - 2006

<u>Certification</u>

Palestine Polytechnic University

PPU

Hebron-Palestine

The Senior Project Entitled:

Establishment of Palestine Polytechnic University (PPU)
BaseLine and Geodetics Network

Prepared by

Ahlam Alan **Ammar Al-Jabari**

Raghad Al-sa'dah

In according with the recommendation of the project supervisors and the acceptance of all examining committee members, this project has been submitted to the department of Civil and Architectural Engineering in the College of Engineering and Technology in partial fulfillment of the requirements of the department for the degree of Bachelor of Engineering.

Project Supervisors
Name:…………….

Department Chairman
Name:……………….

Committee member's signature

Name:……………. **Name:…………….** **Name:…………….**

June - 2006

إهداء

إلى الذين كان لهم أثر فى مداد هذا القلم .....

إلى الذين رفعوا أيديهم لقيوم السموات والأرض .....

..... داعين لنا بالتوفيق .....

إلى الشمعة التي أضاءت لي طريق الأمل .....

إلى من ضحت بحياتها لتراني أفضل الناس .....

.... إلى أمي ..... الحب والحنان .....

إلى من أضاء لي طريق العلم و المعرفة ..... دون مقابل .....

إلى من له الفضل في وضعي أول الطريق ..... إلى أبي ...

إلى من سهروا بجانبي ..... وكان لهم الفضل في نجاحي ...

... أخوتي وأخواتي .....

إلى من حثنا للولوج في طريق النور .....

..... طريق الدعوة الى الله .....

إلى الذين كانوا ولازالوا سائرين في نشر الخير..... والهداية للناس .....

إلى الذين أدعو الله من كل قلبي أن يرزقهم الفردوس الأعلى .....

إلى كل صديق دون إستثناء .....

إلى من فرح لفرحي وحزن لحزني .....

نهدي هذا البحث...

فريق البحث...

شكر و تقدير

إن الشكر و المنة لله وحده دائما الذي لايحمد على مكروه سواه

ونتقدم بجزيل الشكر وأسمى آيات التقدير، إلى كل الذين ساهموا ووقفوا معنا من أجل تحقيق هدفنا المنشود في إنجاز هذا البحث المتواضع ليضعونا على أول الطريق ... طريق مواجهة الحياة العملية، ونخص بالذكر جامعتنا الغالية جامعة بوليتكنك فلسطين وكلية الهندسة و التكنولوجيا و دائرة الهندسة المدنية و المعمارية وأساتذتنا الأفاضل، ونخص بالذكر مشرفنا العلمي الأستاذ المهندس فيضي شبانة الذي لم يأل جهدا في ولادة هذا البحث إلى النور عبر توجيهاته وإرشاداته العلمية والعملية البناءه، ومتابعة خطواتنا أولا بأول، والى الدكتور ماجد ابو شرخ و الاستاذ ماهر العويوي لما كان لهما من أثر عبر توجيهاتهما العلمية والفنية ، و إلى آبائنا وأمهاتنا الذين سهروا الليالي وكابدوا من أجل راحتنا، وتحقيق أحلامنا، والنهوض بنا إلى مصاف أهل العلم..والى الأستاذ المهندس عطا العطاونة والى الأستاذ المهندس نضال أبو رجب لما قدماه لنا من مساعدة ونصح ، والى كل الذين لم نذكرهم حصرا لهم متسع في القلب أيضا .

لكم منا مرة أخرى أسمى آيات الشكر والمحبة طالما حيينا.

وتفضلوا منا بقبول فائق الاحترام ...

فريق البحث...

ABSTRACT

Establishment of Palestine Polytechnic University (PPU) BaseLine and Geodetic Network

Prepared by

Ahlam Alan **Ammar Al-Jabari**

Raghad Al-sa'dah

Palestine Polytechnic University

SUPERVISED BY

Eng. FAYDI SHABANEH

This project can be divided in two parts:

Firstly Establishment of PPU baseline which can be useful for the application of geomatics not only for practical courses but also for serving local community in the future, this part is finished by using the total station instrument and GPS techniques all the corrections introduced to the base line in order to obtain the most probable value of it.

Secondly built the PPU geodetic network, the first part of this network built in this project which includes connected PPU buildings and Al-Shareya school using GPS technique static, with all corrections needed for the GPS observations.

ملخص

Establishment of Palestine Polytechnic University (PPU)

Base Line and Geodetics Network

إعداد

أحلام علان **عمار الجعبري**

رغد السعدة

جامعة بوليتكنيك فلسطين

إشراف

م. فيضي شبانة

قمنا بإنشاء **خط أساس باستخدام** (GPS, Total station)**، وعملنا** جميع التصحيحات **اللازمة للاستفادة** منه في المشاريع القادمة.

قمنا بإنشاء شبكة جيوديسية، تربط بين مباني جامعة بوليتكنك فلسطين **مع مبنى المدرسة** الشرعية **للبنين**، **باستخدام** (GPS) بطريقة **الرصد الثابت** (Static) **وعملنا** جميع التصحيحات للحصول على شبكة دقيقة**،** هذا وقد تم ادخال جميع التصحيحات اللازمة على هذة النقاط.

TABLE OF CONTENTS

CHAPTER 2 INTRODUCTION OF GPS AND TOTAL STATION

CHAPTER 3 ERROR IN GPS AND TOTAL STATION

CHAPTER 4 BASELINE AND GEOMATIC NETWORKS

CHAPTER 5 APPLICATOIN OF LEAST SQUARES

CHAPTER 6 MEASUREMENTS AND CALCALATION

CHAPTER 7 CONCLUSION AND RECOMMENDATION

REFERENCES

APPENDIX

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREEVIATIONS

A-S:	Anti Spoofing
C/A:	Coarse Acquisition
DoD:	Department of Defense
DOP:	Dilution of Precision
EDM:	Electronic Distance Measuring
GDOP:	Geometric Dilution of Precision
GIS:	Geographic Information Systems
GPS:	Global Positioning System
HDOP:	Horizontal Dilution of Precision
LSS:	Least Square Solution
MSL:	Mean Sea Level
NG:	National Grid
No:	Number
P- Code:	Precision Code
PDOP:	Positional Dilution of Precision
PPM:	Part Per Million
PPU:	Palestine Polytechnic University
RTK:	Real Time Kinematics
S/A:	Selective Availability
U. S:	United state
UTM:	Universal Transverse Mercator
VCV:	Variance-Covariance
VDOP:	Vertical Dilution of Precision
WGS84:	World Geodetic System 1984
2D:	Two Dimension
3D:	Three Dimension

CHAPTER

1

INTRODUCTION

The following contents are going to be covered in this chapter:

1.1 Introduction

1.2 Project importance

1.3 Project objectives

1.4 Work Methodology

1.5 Project outline

1.6 Problems

1.7 Previous studies

1.8 Project Time Table "Schedule"

1.9 Study Area

CHAPTER ONE

Introduction

1.1 Introduction

GPS: The Global Positioning System (GPS) is a satellite-based navigation system operated by the U.S. Department of Defense (DoD). GPS provides all weather, worldwide, 24-hour position and time information.

Baseline: The position of a point relative to another point. In GPS surveying, this is the position of one receiver relative to another. When the data from these two receivers is collected and computed using carrier-phase processing, the result is a baseline comprising a three-dimensional vector between the two stations.

Geodetic Control point: The impact of GPS on geodetic control surveying, since its introduction in the 1980's, is nothing short of astounding. GPS has put into the hands of the surveying community a powerful tool for the establishment of precise geodetic control. Just as electronic distance measurement and desk top computers profoundly influenced the practice of land surveying, GPS has contributed to the profession's evolution. This evolution has brought with it reduction in cost and an increased demand for geodetic control services.

1.2 Project importance

- Establishment the baseline which can useful for the application of geomatics not only for practical courses but also for serving local community in the future.
- Geodetic application, including the establishment of control networks.

1.3 Project objectives

- Get enough training on the new receiver in PPU labsTrimble 5700.
- Comparison between surveying techniques using GPS.
- Check coordinate in one or more national grid points.
- Establishing geodetic control in PPU Campus and Al-Shareya school
- Supporting engineering construction.
- Establishing baseline between buildings (A&B) in Wadi Al-Hareyya.

1.4 Work Methodology

- Firstly, two control points were chosen in Wadi alhareyya between Building A and Building B; to establish the baseline. These points connected to the national grid.
- Secondly, the most probable value of horizontal distance and height difference between these two control points were measured using the appropriate equipment observation using trilateration method were commenced. Coordinate of these points were calculated and distance were calibrated by total station.

- GPS instrument were used to determine the ellipsoidal height of these two points and to check there coordinates, and then geoid ellipsoid separation can be determined.
- In the second part of the project, the places of the geodetic points were selected and observed by using GPS technique static.
- Finally, the results of observations must be processed using Least Squares techniques.

1.5 Project out line

- Introduction.
- GPS and Total Station.
- Errors in GPS and Total Station.
- Baseline and Geomatic Networks.
- Application of Least Squares Solution of the Geomatics.
- Measurements and Calculations.
- Conclusions and Recommendations.
- Appendix
- References

1.6 Problem

The only problem was the atmospheric conditions:

The problems causes the project work consuming much times, as the field survey were postponed several time because of rains and bad weather conditions especially in 1st semester.

1.7 Previous studies

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

1.8 Project Time Table "Schedule"

Table (1.1): Project Schedule for course I in year 2005 - 2006

stages	Week No.	Time frame in weeks													
		1	2	3	4	5	6	7	8	9	10	11	12	13	14
Choosing the project	2														
Reconnaissance	1														
Preparing the first three chapters	2														
Initial land survey	4														
First Office calculations	2														
First preparation of project introduction report	1														
project introduction report revision	1														
Final project introduction report printing and handing	1														

Table (1.2): Project Schedule for course II in year 2005 - 2006

stages	**Week No.**	**Time frame in weeks**													
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**
Completing field work "land survey"	3														
Completing office calculations	5														
Preparing map revision chapters	2														
Preparing theory chapters	4														
Preparing field survey chapter	2														
Preparing calculations and analysis chapter	3														
Conclusions and recommendations	1														
Final revision and printing all chapters	2														
Handing the final report of project	1														

1.9 Study Area

The location of the study area in our project was in the following parts of Hebron city: Wadi Al Hareya, Gabal Abu Rumman, Wadi Abu Kteilah, and Ein Sara, we established geodetic network to connect Palestine Polytechnic University buildings and Al-Shareyya school for boys, figure 1.1.

Stations A and B are control stations, and stations C, D, E, and F are points of unknown position.

A: Point above Building A in Wadi Al-Hareya

B: Point above Building B in Wadi Al-Hareya

C: Point above Building Abu Rumman (Friends of Fawzikawah IT Center of Excellence)

D: Point above Al-Shareya school for boys

E: Point above Building A in Abu Kteilah

F: Point above main Building Ein Sara (PPU)

* For more information please see the next five pages

CHAPTER

2

INTRODUCTION OF GPS AND TOTAL STATION

The following contents are going to be covered in this chapter:

CHAPTER TWO

INTRODUCTION OF GPS AND TOTAL STATION

2.1 What is GPS?

The Global Positioning System (GPS) is a location system based on a constellation of about 24 satellites orbiting the earth at altitudes of approximately 11,000 miles. GPS was developed by the United States Department of Defense (DoD), for its tremendous application as a military locating utility. The DoD's investment in GPS is immense. Billions and billions of dollars have been invested in creating this technology for military uses. However, over the past several years, GPS has proven to be a useful tool in non-military mapping applications as well.

GPS satellites are orbited high enough to avoid the problems associated with land based systems, yet can provide accurate positioning 24 hours a day, anywhere in the world. Uncorrected positions determined from GPS satellite signals produce accuracies in the range of 50 to 100 meters. When using a technique called differential correction, users can get positions accurate to within 5 meters or less.

Today, many industries are leveraging off the DoD's massive undertaking. As GPS units are becoming smaller and less expensive, there are an expanding number of applications for GPS. In transportation applications, GPS assists pilots and drivers in pinpointing their locations and avoiding collisions. Farmers can use GPS to guide equipment and control accurate distribution of fertilizers and other chemicals. Recreationally, GPS is used for providing accurate locations and as a navigation tool for hikers, hunters and boaters.

Many would argue that GPS has found its greatest utility in the field of Geographic Information Systems (GIS). With some consideration for error, GPS can provide any point on earth with a unique address (its precise location). A GIS is basically a descriptive database of the earth (or a specific part of the earth). GPS tells you that you are at point X,Y,Z while GIS tells you that X,Y,Z is an oak tree, or a spot in a stream with a pH level of 5.4. GPS tells us the "where". GIS tells us the "what". GPS/GIS is reshaping the way we locate, organize, analyze and map our resources.

2.2 How the Current Locations of GPS Satellites are Determined?

GPS satellites are orbiting the Earth at an altitude of 11,000 miles. The DoD can predict the paths of the satellites vs. time with great accuracy. Furthermore, the satellites can be periodically adjusted by huge land-based radar systems. Therefore, the orbits, and thus the locations of the satellites, are known in advance. Today's GPS receivers store this orbit information for all of the GPS satellites in what is known as an almanac. Think of the almanac as a "bus schedule" advising you of where each satellite will be at a particular time. Each GPS satellite continually broadcasts the almanac. Your GPS receiver will automatically collect this information and store it for future reference.

The Department of Defense constantly monitors the orbit of the satellites looking for deviations from predicted values. Any deviations (caused by natural atmospheric phenomenon such as gravity), are known as ephemeris errors. When ephemeris errors are determined to exist for a satellite, the errors are sent back up to that satellite, which in turn broadcasts the errors as part of the standard message, supplying this information to the GPS receivers.

By using the information from the almanac in conjunction with the ephemeris error data, the position of a GPS satellite can be very precisely determined for a given time.

2.3 Computing the Distance between Your Position and the GPS Satellites

GPS determines distance between a GPS satellite and a GPS receiver by measuring the amount of time it takes a radio signal (the GPS signal) to travel from the satellite to the receiver. Radio waves travel at the speed of light, which is about 186,000 miles per second. So, if the amount of time it takes for the signal to travel from the satellite to the receiver is known, the distance from the satellite to the receiver (distance = speed x time) can be determined. If the exact time when the signal was transmitted and the exact time when it was received are known, the signal's travel time can be determined.

In order to do this, the satellites and the receivers use very accurate clocks which are synchronized so that they generate the same code at exactly the same time. The code received from the satellite can be compared with the code generated by the receiver. By comparing the codes, the time difference between when the satellite generated the code and when the receiver generated the code can be determined. This interval is the travel time of the code. Multiplying this travel time, in seconds, by 186,000 miles per second gives the distance from the receiver position to the satellite in miles.

2.4 Four (4) Satellites to give a 3D position

Three measurements can be used to locate a point, assuming the GPS receiver and satellite clocks are precisely and continually synchronized, thereby allowing the distance calculations to be accurately determined. Unfortunately, it is impossible to synchronize these two clocks, since the clocks in GPS receivers are not as accurate as the very precise and expensive atomic clocks in the satellites. The GPS signals travel from the satellite to the receiver very fast, so if the two clocks are off by only a small fraction, the determined position data may be considerably distorted.

The atomic clocks aboard the satellites maintain their time to a very high degree of accuracy. However, there will always be a slight variation in clock rates from satellite to satellite. Close monitoring of the clock of each satellite from the ground permits the control station to insert a message in the signal of each satellite which precisely describes the drift rate of that satellite's clock. The insertion of the drift rate effectively synchronizes all of the GPS satellite clocks.

The same procedure cannot be applied to the clock in a GPS receiver. Therefore, a fourth variable (in addition to x, y and z), time, must be determined in order to calculate a precise location. Mathematically, to solve for four unknowns (x, y, z, and t), there must be four equations. In determining GPS positions, the four equations are represented by signals from four different satellites.

2.5 GPS Measuring Techniques

There are several measuring techniques that can be used by most GPS Survey Receivers. The surveyor should choose the appropriate technique for the application.

Static Surveys

This was the first method to be developed for GPS surveying. It can be used for measuring long baselines (usually 20km (16 miles) and over). One receiver is placed on a point whose coordinates are known accurately in WGS84. This is known as the Reference Receiver. The other receiver is placed on the other end of the baseline 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 at each station. The data collection rate may be typically set to 15, 30 or 60 seconds.

The receivers have to collect data for a certain length of time. This time is influenced by the length of the line, the number of satellites observed and the satellite geometry (dilution of precision or DOP). As a rule of thumb, the observation time is a minimum of 1 hour for a 20km line with 5 satellites and a prevailing GDOP of 8. Longer lines require longer observation times.

Once enough data has been collected, the receivers can be switched off. The Rover can then be moved to the next baseline and measurement can once again commence. It is very important to introduce redundancy into the network that is being measured. This involves measuring points at least twice and creates safety checks against problems that would otherwise go undetected.

A great increase in productivity can be realized with the addition of an extra Rover receiver. Good coordination is required between the survey crews in order to maximize the potential of having three receivers.

*An example [1] is the network ABCDE (sea figure 2.1) has to be measured with three receivers. The coordinates of A are known in WGS84. The receivers are placed on A, B and C. GPS data is recorded for the required length of time.

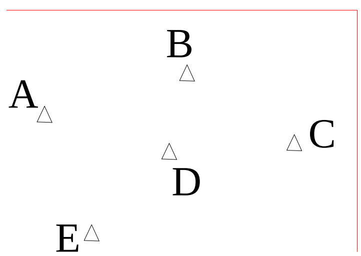


Figure (2.1): The network

After the required length of time, the receiver that was at E moves to D and B moves to C. The triangle ACD is measured. (See figure 2.2)

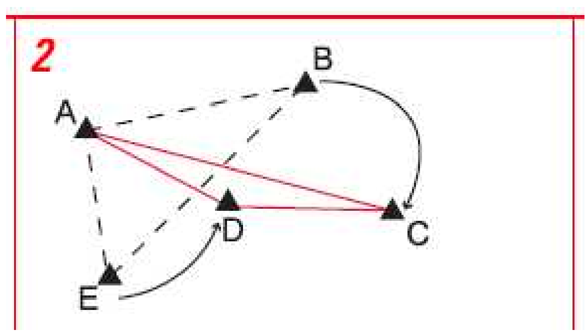


Figure (2.2): The receiver moves

[1] From reference No. 12

Then A moves to E and C moves to B. The triangle BDE is measured. (See figure 2.3)

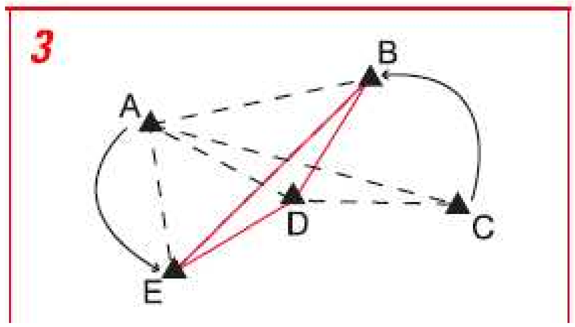


Figure (2.3): The receiver moves

Finally, B moves back to C and the line EC is measured. (See figure 2.4)

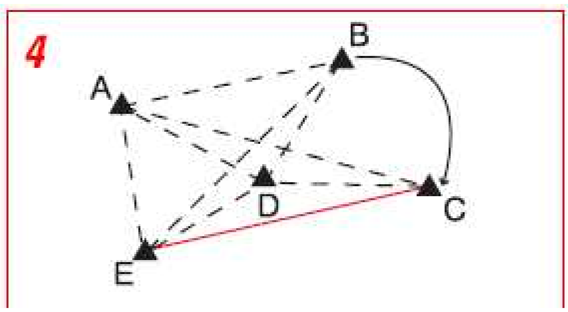


Figure (2.4): The line EC is measured

The end result is the measured network ABCDE. One point is measured three times and every point has been measured at least twice. This provides redundancy. Any gross errors will be highlighted and the offending measurement can be removed. (See figure 2.5)

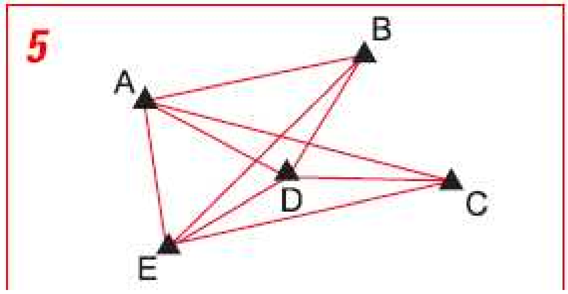


Figure (2.5): Finally the measured network

Rapid Static - Used for establishing local control networks, Network densification etc. Offers high accuracy on baselines up to about 20km and is much faster than the Static technique.

Kinematics - Used for detail surveys and measuring many points in quick succession. Very efficient way of measuring many points that is close together. However, if there are obstructions to the sky such as bridges, trees, tall buildings etc., and less than 4 satellites are tracked, the equipment must be reinitialized which can take 5-10 minutes.

RTK - Real Time Kinematics 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 applications as Kinematics. 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 radio sources and also line of sight blockage.

2.6 Introduction of the Total Station

A total station is a combination electronic transit and electronic distance measuring device (EDM). With this device, as with a transit and tape, one may determine angles and distances from the instrument to points to be surveyed. With the aid of trigonometry, the angles and distances may be used to calculate the actual positions (x, y, and z or northing, easting and elevation) of surveyed points in absolute terms.

A standard transit is basically a telescope with cross-hairs for sighting a target; the telescope is attached to scales for measuring the angle of rotation of the telescope (normally relative to north as 0 degrees) and the angle of inclination of the telescope (relative to the horizontal as 0 degrees). After rotating the telescope to aim at a target, one may read the angle of rotation and the angle of inclination from a scale. The electronic transit provides a digital read-out of those angles instead of a scale; it is both more accurate and less prone to errors arising from interpolating between marks on the scale or from mis-recording. The readout is also continuous; so angles can be checked at any time.

2.7 Applications of the Total Station

- Layout of control points on or offset to construction lines.
- Checking or tying in to property boundaries
- Layout of excavation lines
- As-built checks
- Layout of construction control lines on concrete pad for subcontractor use
- Light topographical measurements for cut/fill balance

CHAPTER

3

ERRORS IN GPS AND TOTAL STATION

The following contents are going to be covered in this chapter:

CHAPTER THREE

ERRORS IN GPS AND TOTAL STATION

3.1 Error Sources in GPS

Up until this point, it has been assumed that the position derived from GPS is very accurate and free of error, but there are several sources of error that degrade the GPS position from a theoretical few meters to tens of meters.

These error sources are:
1. Ionosphere and atmospheric delays
2. Satellite and Receiver Clock Errors
3. Multipath
4. Dilution of Precision
5. Selective Availability (S/A)
6. Anti Spoofing (A-S)

3.1.1. Ionosphere and Atmospheric delays

As the satellite signal passes through the ionosphere, it can be slowed down, the effect being similar to light refracted through a glass block. These atmospheric delays can introduce an error in the range calculation as the velocity of the signal is affected. (Light only has a constant velocity in a vacuum).

The ionosphere does not introduce a constant delay on the signal. There are several factors that influence the amount of delay caused by the ionosphere. (See Figure 3.1)

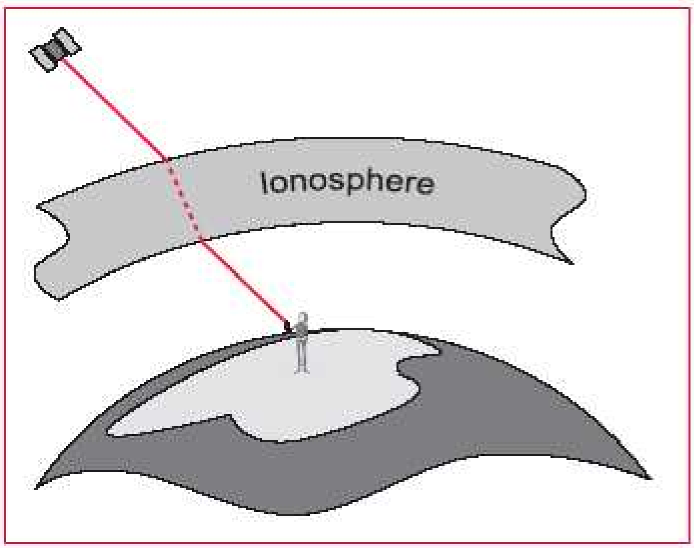


Figure (3.1): Ionosphere error [1]

a. Satellite elevation. Signals from low elevation satellites will be affected more than signals from higher elevation satellites. This is due to the increased distance that the signal passes through the atmosphere. (See Figure 3.2)

[1] From reference No. 12

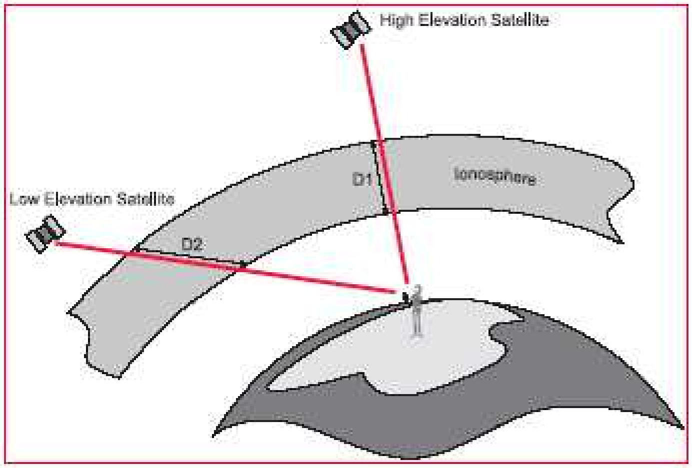


Figure (3.2): Satellite elevation [1]

b. The density of the ionosphere is affected by the sun. At night, there is very little ionosphere influence. In the day, the sun increases the effect of the ionosphere and slows down the signal.

The amount by which the density of the ionosphere is increased varies with solar cycles (sunspot activity). Sunspot activity peaks approximately every 11 years.

In addition to this, solar flares can also randomly occur and also have an effect on the ionosphere. Ionosphere errors can be mitigated by using one of two methods:

- The first method involves taking an average of the effect of the reduction in velocity of light caused by the ionosphere. This correction factor can then be applied to the range calculations. However, this relies on an average and obviously this average condition does not occur all of the

[1] From reference No. 12

time. This method is therefore not the optimum solution to Ionosphere Error mitigation.

- The second method involves using .dual-frequency. GPS receivers. Such receivers measure the L1 and the L2 frequencies of the GPS signal. It is known that when a radio signal travels through the ionosphere it slows down at a rate inversely proportional to its frequency. Hence, if the arrival times of the two signals are compared, an accurate estimation of the delay can be made. Note that this is only possible with dual frequency GPS receivers. Most receivers built for navigation are single frequency.

c. Water Vapour also affects the GPS signal. Water vapor contained in the atmosphere can also affect the GPS signal. This effect, which can result in position degradation, can be reduced by using atmospheric models.

3.1.2 Satellite and Receiver clock errors

Even though the clocks in the satellite are very accurate (to about 3 nanoseconds) they do sometimes drift slightly and cause small errors, affecting the accuracy of the position. The US Department of Defense monitors the satellite clocks using the Control Segment and can correct any drift that is found.

3.1.3 Multipath Errors

Multipath occurs when the receiver antenna is positioned close to a large reflecting surface such as a lake or building. The satellite signal does not travel directly to the antenna but hits the nearby object first and is reflected into the antenna creating a false measurement.

Multipath can be reduced by use of special GPS antennas that incorporate a ground plane (a circular, metallic disk about 50cm (2 feet) in diameter) that prevent low elevation signals reaching the antenna. (See Figure 3.3)

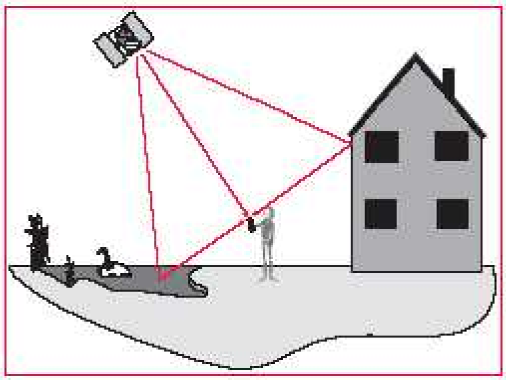


Figure (3.3): Multipath error [1]

For highest accuracy, the preferred solution is use of a choke ring antenna. A choke ring antenna has 4 or 5 concentric rings around the antenna that trap any indirect signals.

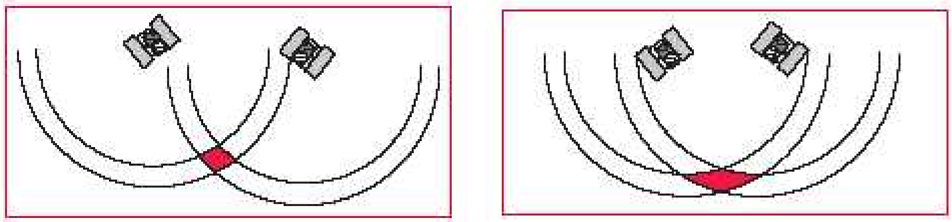
Multipath only affects high accuracy, survey type measurements. Simple handheld navigation receivers do not employ such techniques.

3.1.4 Dilution of Precision

The Dilution of Precision (DOP) is a measure of the strength of satellite geometry and is related to the spacing and position of the satellites in the sky.

[1] From reference No. 12

The DOP can magnify the effect of satellite ranging errors. (See Figure 3.4) The principle can be best illustrated by diagrams:



(a) Well spaced satellites – Low uncertainty of position

(b) Poorly spaced satellites – high uncertainty of position

Figure (3.4): Dilution of Precision [1]

The range to the satellite is affected by range errors previously described. When the satellites are well spaced, the position can be determined as being within the shaded area in the diagram and the possible error margin is small. When the satellites are close together, the shaded area increases in size, increasing the uncertainty of the position.

Different types of Dilution of Precision or DOP can be calculated depending on the dimension:

VDOP: Vertical Dilution of Precision. Gives accuracy degradation in vertical direction.

HDOP: Horizontal Dilution of Precision. Gives accuracy degradation in horizontal Direction.

PDOP: Positional Dilution of Precision. Gives accuracy degradation in 3D position.

GDOP: Geometric Dilution of Precision. Gives accuracy degradation in 3D position and Time.

[1] From reference No. 12

The most useful DOP to know is GDOP since this is a combination of all the factors. Some receivers do however calculate PDOP or HDOP which do not include the time component.

The best way of minimizing the effect of GDOP is to observe as many satellites as possible. Remember however, that the signals from low elevation satellites are generally influenced to a greater degree by most error sources.

Mask angles

As a general guide, when surveying with GPS it is best to observe satellites that are 15° above the horizon. The most accurate positions will generally be computed when the GDOP is low, (usually 6 or less).

3.1.5 Selective Availability (S/A)

This is an artificial error introduced into the satellite data by the US DoD to reduce the possible accuracy of a position to 100 meters for commercial users. Without SA typical C/A code (commercial) GPS position accuracy would be about 20 meters. However to deny this accuracy to potential hostile users, the US Defense Department has the ability to corrupt the satellite's signals to degrade the accuracy to as much as 100 meters. In addition to this, other sources of error include:

- Satellite clock and ephemeris errors
- Environmental errors caused by atmospheric sources
- Receiver induced errors

The first two of these sources of error are removed by the use of DGPS which improves the accuracy of the GPS positions that are obtained to about 10 meters depending on the GPS receiver used and the configuration of the satellite constellation.

3.1.6 Anti-Spoofing (A-S)

Anti-Spoofing is similar to S/A in that it's intention is to deny civilian and hostile powers access to the P-code part of the GPS signal and hence force use of the C/A code which has S/A applied to it.

3.2 Errors in Total Station

3.2.1 Electromagnetic waves in Total Station

Electromagnetic waves can be represented by a sinusoidal wave motion. The number of times in 1 second that a wave completes a cycle is called the frequency (f), and is measured in Hz.

The length of one cycle is called the wavelength (ª), which can be determined as a function of the frequency from

$$\lambda = \frac{v}{f} \qquad \ldots\ldots\ldots\ (3.1)$$

Where: *v* is the speed of propagation of the wave.

The speed of electromagnetic waves in a vacuum is called the speed of light, *c*, and is taken to be 299,792,458 m s-1. The accuracy of an EDM instrument depends ultimately on the accuracy of the estimated velocity of the electromagnetic wave through the atmosphere.

A relationship expressing the instantaneous amplitude of a sinusoidal wave is

$$A = A_{max} \sin\varphi + A_0 \quad \ldots\ldots\ldots (3.2)$$

Where: A_{max} is the maximum amplitude developed by the source, A_0 is the reference amplitude, and φ is the phase angle which completes a cycle in 2π radians or 360°.

In an EDM system, distance is measured by the difference in phase angle between the transmitted and received versions of a sinusoidal wave. The double path length (2D) between instrument and reflector is the distance covered by the radiation from an EDM measurement. It can be represented in terms of the wavelength of the measuring unit:

$$2D = n\lambda_m + \Delta\lambda_{m2} \quad \ldots\ldots\ldots (3.3)$$

The distance from instrument to reflector is *D*, λ_m is the wavelength of the measuring unit, *n* is the integer number of wavelengths traveled by the wave, and $\Delta\lambda_m$ is the fraction of the wavelength traveled by the wave. Therefore, the distance *D* is made of two separate elements. An EDM instrument using continuous electromagnetic waves can only determine $\Delta\lambda_m$ by phase comparison (Figure3. 5).

If the phase angle of the transmitted wave measured at the instrument is φ_1, and the phase angle measured on receipt is φ_2, then

$$\Delta\lambda_m = \frac{\lfloor \varphi_2 - \varphi_1 \rfloor}{2\pi} \lambda_m \quad \ldots\ldots\ldots (3.4)$$

The phase angle ¤$_2$ can apply to any incoming wavelength, so phase comparison will only provide a determination of the fraction of a wavelength traveled by the wave, leaving the total number, *n*, ambiguous.

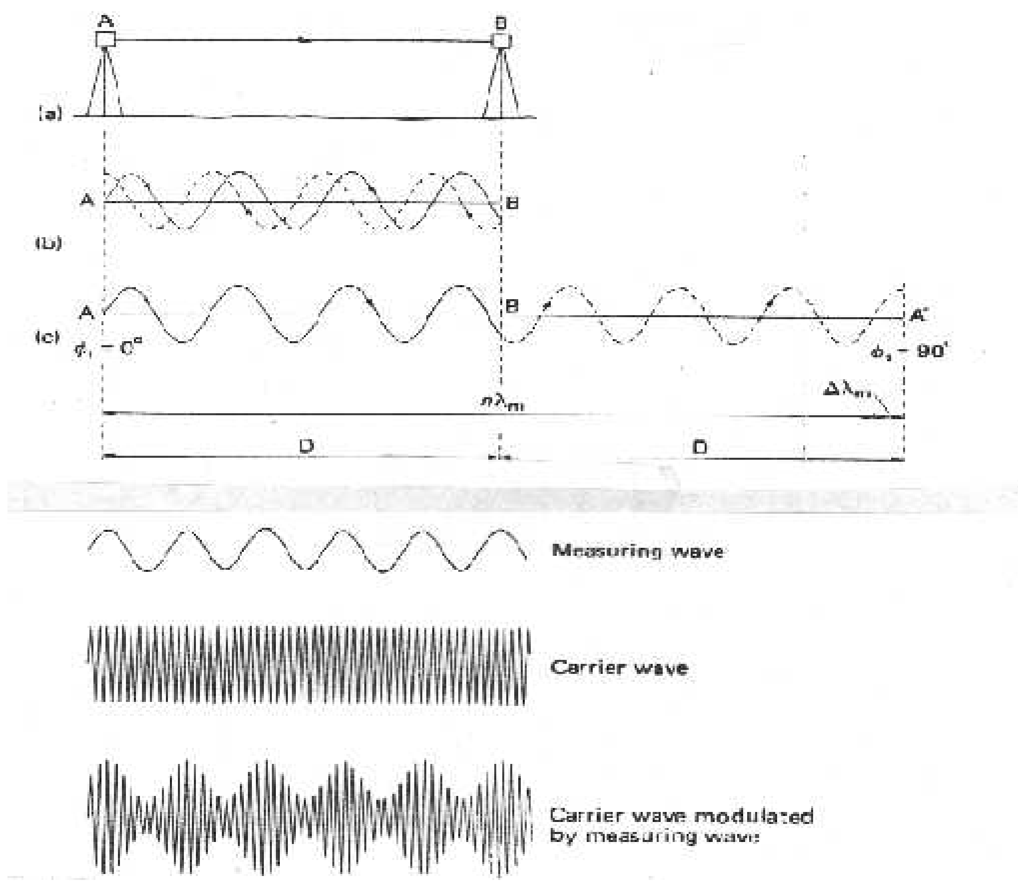


Figure (3.5): Phase comparison [1]

[1] From reference No. 9

From figure (3.5):

a. An EDM is set up at A and a reflector at B for determination of the slope length (D). During measurement, an electromagnetic wave is continuously transmitted from A towards B where it is reflected back to A.

b. The electromagnetic wave path from A to B has been shown, and for clarity, the same sequence is shown in (c), but the return wave has been opened out.

c. Points A and A' are effectively the same, since the transmitter and receiver would be side by side in the same unit at A. The lowermost portion also is lustrates the ideal of modulation of the carrier wave by the measuring wave.

For many EDM instruments, accuracy in measurement between 1 and 10 mm is specified at short ranges, and a phase resolution of 1 in 10,000 is normal. Assuming an accuracy of at least 1 mm, therefore, a measuring wavelength (a_m) of 10 m is required. Approximating the speed of propagation, v, by 3 x 10^8 m s^{-1}, 10 m corresponds to a frequency of 30 MHz. Adequate propagation of an electromagnetic signal of 30 MHz frequency for EDM purposes is not practical, so a higher frequency carrier wave is used and modulated by the measuring wave (Figure 3.5). In the case of our instrument, the carrier wave is infra-red, with a wavelength of 0.835 µm, corresponding to a frequency of 3.6 x 10^5 GHz.

The return signal is usually amplified, and then the phase difference is determined digitally. The signal derived from the modulation triggers off the counting mechanism every time the signal changes from negative to positive. The signal derived from the reflected ray stops the counting mechanism. For our instrument, the integer ambiguity (n) of wavelengths is determined using the coarse measurement frequency of 74,927 Hz, equivalent to 2000 m, and the amplitude modulation of 4,870,255 Hz (30.7692 m) provides the fine measurement (Δa_m). In order to achieve the stated accuracy of 5 mm, the phase measurements are accurate to 1 part in 6,154.

3.2.2 Errors in Distance Measurement in Total Station

The path of electromagnetic energy is the true distance that is measured by the EDM, and will be determined by the variability in the refractive index through the atmosphere.

$$\text{Refractive index (k)} = \frac{\text{velocity in vacuum}}{\text{velocity in medium}} = \frac{c}{v} \quad \ldots\ldots\ldots (3.5)$$

Since the medium is air, the velocity is nearly the same as that of a vacuum, and so the refractive index is nearly one, and for standard conditions may be taken to be 1.000320. The exact value of the refractive index is dependent on the atmospheric conditions of temperature, pressure, water vapor pressure, frequency of the radiated signal, and composition. Therefore, for measurements of the highest accuracy, adequate atmospheric observations must be made. Other potential error sources are from path curvature (similar to the curvature of the earth).

Scale error

One source of error that is adjusted for in our instrument is a scale correction in units of ppm that adjusts for slight errors in the reference frequency and in the accuracy of the average group refractive index along the line of measurement. The ppm value is set to 0 on the EDM, and adjusted in the theodolite.

Atmospheric correction is one scale error adjustment that takes into account both atmospheric pressure and temperature. It is an absolute correction for the true velocity of propagation, and not a relative scale correction like reduction to sea level. To determine the atmospheric correction to an accuracy of 1 ppm, measure the ambient temperature to an accuracy of 1°C and atmospheric pressure to 3 mb.

For most applications, and approximate value for the atmospheric correction (within about 10 ppm) is adequate. This can be obtained by taking the average temperature for the day and the height above mean sea level of the survey site. A temperature change of about 10°C or a change in height above sea level of about 350 m (= 35 mb) varies the scale correction by only 10 ppm. The atmospheric correction is computed in accordance with the following formula:

$$\Delta D = 281.8 - 0.29065 P_1 + 0.00366 T \quad \text{……….} \ (3.6)$$

Where:

ΔD_1 = atmospheric correction (ppm),

P = atmospheric pressure (mb),

T = ambient temperature (°C).

For extreme conditions of 30°C temperature change and 100 mb pressure change, one can expect variations in scale error of 50 ppm in a day. This maximum value is ten times greater than the stated initial accuracy of the instrument, and therefore should be accounted for by adjusting the scale error on the theodolite occasionally during the day's surveying effort.

The correction in ppm for the reduction to mean sea level is based upon the formula:

$$\Delta D^2 = -10^3 \frac{H}{R} \text{ ………. (3.7)}$$

Where:

ΔD^2 = reduction to MSL in ppm, H = height of EDM above MSL, and R = 6378 km (earth radius).

This correction is a constant and should be determined at the beginning of the survey effort.

3.3 Errors propagation in distance by Total Station

Systematic errors in distances measured by Total Station are caused by:

(a) The effects of atmospheric conditions on propagated wave velocities of light waves and electromagnetic waves in air.

(b) The differences between the effective centers of the transmitter and reflector and their respective plump lines.

(c) Transmitter nonlinearity.

Atmospheric conditions are observed allowing corrections to be made for (a) and the system are calibrated to permit determination of constants to correct for (b) and (c).

Comment will focus on short-range, light_ wave instrument, where the distances that remain in distances measured by Total Station are the small uncertainties in the:

(1) Modulation frequency, f, that differ from theoretical value.
(2) Index of refraction caused by variation in atmospheric conditions along line measured.
(3) Determination of partial wavelengths by phase comparison.
(4) Calibration of the system constant to account for systematic errors from (b) and (c) above.
(5) Centering of the Total Station transmitter and reflector over their respective station.

Effect of the errors due to uncertainties (1) and (2) are obtained by propagating them through an equation for distance.

$$D = \frac{1}{2}\left(\frac{mV_0}{nf} + d\right) \quad \text{……….} \ (3.9)$$

Applying the law of error propagation to equation (3.9), neglecting the second term, and taking $D \approx (mV_0)/2nf$ yields

$$\sigma_{D1} = \sqrt{\left(\frac{(\sigma_r)^2}{(n)^2} + \frac{(\sigma_f)^2}{(f)^2}\right) D^2} \quad \text{……….} \ (3.10)$$

in which $\pm_{D1}$ equals the standard deviation in the measured distance due to uncertainties in the refractive index, $\pm_{r}$, and the modulation frequency, $\pm_{f}$, Note that these two sources of error are proportional to the distance measured.

Independent errors in phase comparison, $\pm_{\text{¤}}$, determination of the system constant, $\pm_{sys}$, and centering of the Total Station and reflector, $\pm_{cent}$ propagate to form

$$\pm_{D2} = \sqrt{\pm^{2}_{C-sys} + \pm^{2}_{\text{¤}} + \pm^{2}_{cent}} \quad \ldots\ldots\ldots\ (3.11)$$

Which, when combined with D1, yields the total standard deviation for the distance measurement of

$$\pm_{D,total} = \sqrt{\pm^{2}_{D1} + \pm^{2}_{D2}} \quad \ldots\ldots\ldots\ (3.12)$$

Equation (3.12) is a rigorous expression for propagating the error in a distance measured by EDM. The manufacturers of Total Station equipment use a similar form to describe the accuracy of their systems where their expression is 3-5 mm + 1-5 ppm that results form a simplification of equation (3.12). The 3-5 mm is independent of distance and corresponds to D2, whereas the 1-5 ppm depends on distance and is analogous to D1, both as in equation (3.12).

3.4 Correction of Distances for Atmospheric Conditions

To correct the observed slope distances for varying atmospheric conditions for light-wave instruments, the temperature (t) and atmospheric pressure (p) are recorded at each end of the line. Some models of EDM systems allow entry of the mean, values of (t) and (p) using the keyboard. The microcomputer in the EDM then automatically computes and applies a correction factor to the measured slope distances. In other EDM systems, the correction term, in parts per million, is determined using the mean observed values of (t) and (p) from a chart or monogram furnished by the manufacturer of the EDM. This correction term then is entered into the EDM via the control panel or a special dial. Some practitioners using these latter models prefer to calculate a correction factor, which then is applied to the measured slope distance. In this case, 0 ppm would be dialed into the EDM prior to measuring the slope distance.

The frequency (f) of a signal is the number of oscillations per second. The wavelength is the length between two successive crests of a sinusoidal wave. The wavelength is equal to the speed of the wave (in this case the speed of light) divided by the frequency.

Temperature is measured in either degrees Fahrenheit or degrees Celsius, and the two are related by:

$$^{O}C = \frac{5}{9} * \left({}^{0}F - 32 \right) \quad \ldots\ldots\ldots \ (3.13)$$

To convert °C to an absolute scale (Kelvin), add 273.2°. Pressure is measured in inches or mm of mercury (Hg), or in bars. One bar is about 750.06 mm of Hg. Normal atmospheric pressure is 1.01325 bar, 760 mm Hg, or 29.92" Hg.

CHAPTER

4

BASELINE AND GEOMATIC NETWORKS

The following contents are going to be covered in this chapter:

4.1 Introduction
4.2 Coordinate System
4.3 Problems with Height
4.4 Map Projections and Plane Coordinates
4.5 Single Baseline - Multi-Baseline Solutions
4.6 GPS Baseline Adjustments
4.7 Reduction grid to ground distance

CHAPTER FOUR

BASELINE AND GEOMATIC NETWORKS

4.1 Introduction

In general, a GPS survey campaign involves the use of a small number of receivers to coordinate a large number of stations. The area of survey operations may span distances of merely a few kilometers (as on an engineering site), to several hundred kilometers, or even thousands of kilometers in the case of geodynamical surveys. A typical GPS survey, such as for mapping or control densification, involves distances of the order of several tens of kilometers. The survey may be carried out using conventional static GPS survey techniques, or the modern "high productivity" techniques. However, the principles of network processing are the same whether a small number of baselines were observed over several days using conventional GPS, or many baselines observed in a matter of a few hours.

In a GPS network survey a number of processing strategies are possible:

- If the number of receivers that have been deployed during an observation session is greater than two, then the appropriate single session processing strategy must take into account multiple baselines.
- As the survey cannot be completed during a single session of observations, a suitable, involving the multi-session processing strategy propagation of the results of one session solution into another (and eventually across the entire network) has to be used.

4.2 Coordinate System

4.2.1 The GPS Coordinate System

Although the earth may appear to be a uniform sphere when viewed from space, the surface is far from uniform. Due to the fact that GPS has to give coordinates at any point on the earth's surface, it uses a geodetic coordinate system based on an ellipsoid. An ellipsoid (also known as a spheroid) is a sphere that has been flattened or squashed.

An ellipsoid is chosen that most accurately approximates to the shape of the earth. This ellipsoid has no physical surface but is a mathematically defined surface.

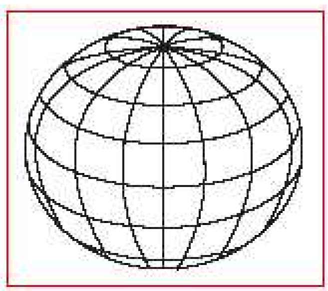


Figure (4.1): An Ellipsoid[1]

There are actually many different ellipsoids or mathematical definitions of the earth's surface, as will be discussed later. The ellipsoid used by GPS is known as WGS84 or World Geodetic System 1984.

A point on the surface of the earth (note that this is not the surface of the ellipsoid), can be defined by using Latitude, Longitude and ellipsoidal height. An alternative method for defining the position of a point is the Cartesian coordinate system, using distances in

[1] From Reference No. 12

the X, Y, and Z axes from the origin or centre of the spheroid. This is the method primarily used by GPS for defining the location of a point in space. (See Figure 4.2)

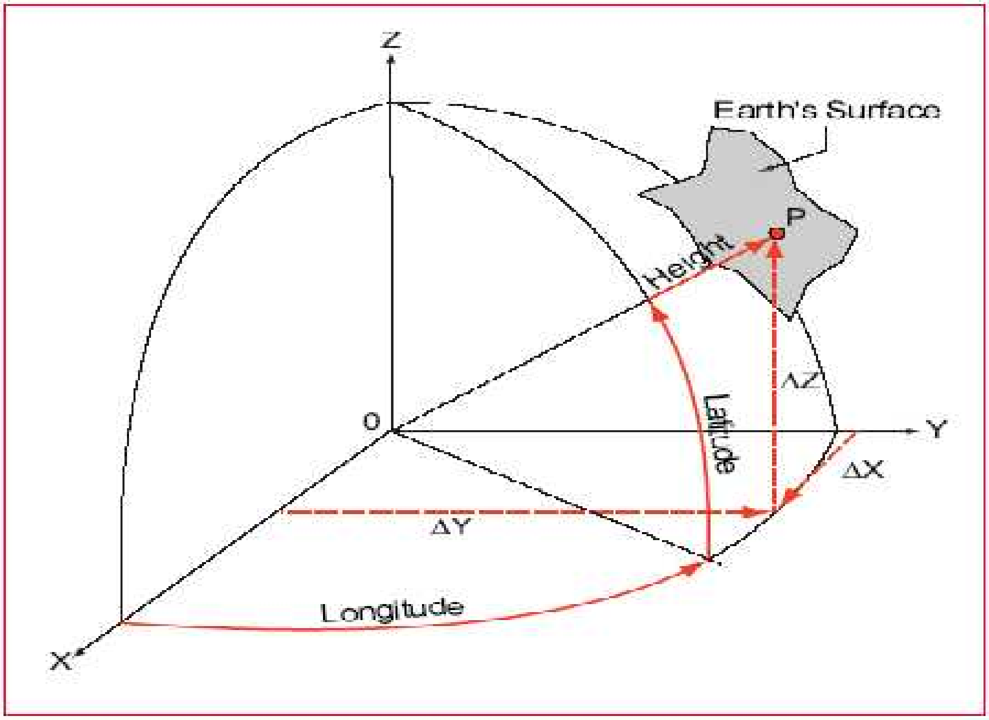


Figure (4.2): Defining coordinates of P by Geodetic and Cartesian coordinates[1]

4.2.2 Local Coordinate Systems

Just as with GPS coordinates, local coordinates or coordinates used in a particular country's maps are based on a local ellipsoid, designed to match the geoids in the area. Usually, these coordinates will have been projected onto a plane surface to provide grid coordinates.

[1] From Reference No. 12

The ellipsoids used in most local coordinate systems throughout the world were first defined many years ago, before the advent of space techniques. These ellipsoids tend to fit the area of interest well but could not be applied to other areas of the earth. Hence, each country defined a mapping system/ reference frame based on a local ellipsoid.

When using GPS, the coordinates of the calculated positions are based on theWGS84 ellipsoid. Existing coordinates are usually in a local coordinate system and therefore the GPS coordinates have to be transformed into this local system.

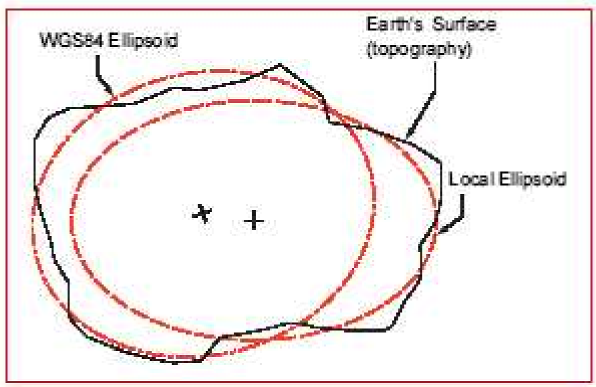


Figure (4.3): The relationship between ellipsoids and the earth's surface[1]

4.3 Problems with Height

The nature of GPS also affects the measurement of height. All heights measured with GPS are given in relation to the surface of the WGS84 ellipsoid. These are known as Ellipsoidal Heights. Existing heights are usually orthometric heights measured relative to mean sea level. Mean sea level corresponds to a surface known as the geoid. The geoid can be defined as an equipotent surface, i.e. the force of gravity is a constant at any point on the geoids.

[1] From Reference No. 12

The geoid is of irregular shape and does not correspond to any ellipsoid. The density of the earth does however have an effect on the geoid, causing it to rise in the more dense regions and fall in less dense regions. The relationship between the geoids, ellipsoid and earth's surface is shown in figure (4.4).

As most existing maps show orthometric heights (relative to the geoids), most users of GPS also require their heights to be orthometric. This problem is solved by using geoidal models to convert ellipsoidal heights to orthometric heights. In relatively flat areas the geoids can be considered to be constant. In such areas, use of certain transformation techniques can create a height model and geoidal heights can be interpolated from existing data. (See Figure 4.4)

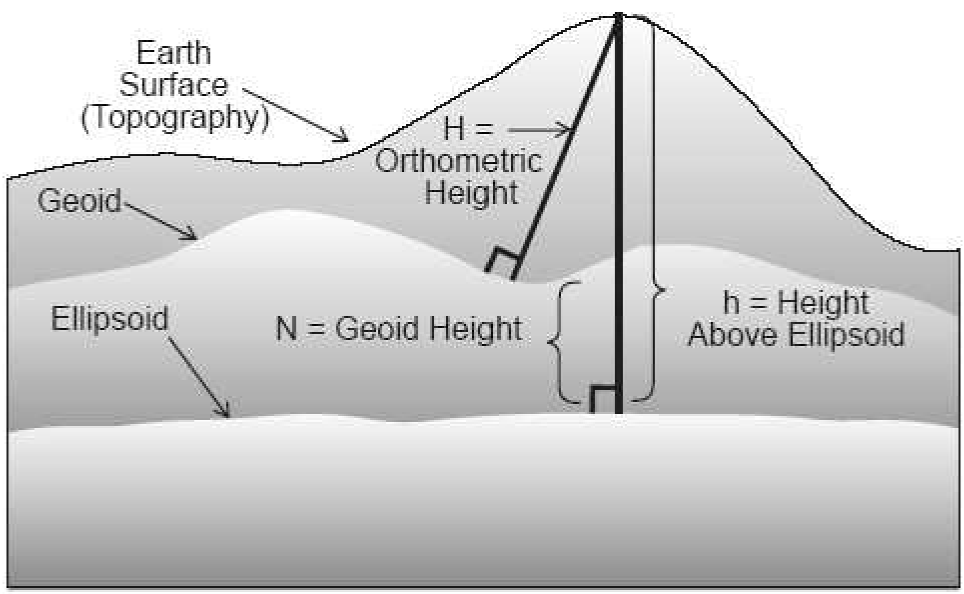


Figure (4.4): Orthometric, Ellipsoid, and Geoid Heights.[1]

[1] From Reference No. 9

GPS Provides Ellipsoid Heights

h = N+H ………. (4.1)

Where:

h = Ellipsoid Height
N = Geoid Height (Geoid Separation)
H = Orthometric Height (elevation)

4.4 Map Projections and Plane Coordinates

Most Surveyors measure and record coordinates in an orthogonal grid system . This means that points are defined by Northing, Easting and orthometric height (height above sea level). Map Projections allow surveyors to represent a 3 dimensional curved surface on a flat piece of paper.

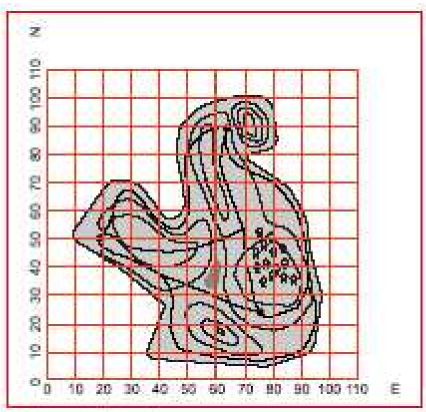


Figure (4.5): A plane grid based map[1]

Such map projections appear as planes but actually define mathematical steps for specifying positions on an ellipsoid in the terms of a plane. The way in which a map

[1] From Reference No. 12

projection generally works is shown in the diagram. Points on the surface of the spheroid are projected on to a plane surface from the origin of the spheroid. The diagram also highlights the problem that it is not possible to represent true lengths or shapes on such a plane. True lengths are only represented where the plane cuts the spheroid (point's c and g). (See Figure 4.6)

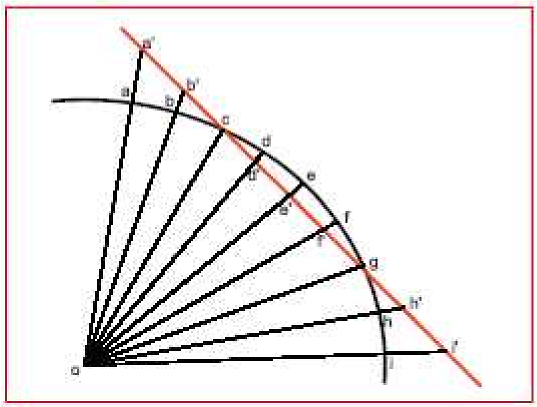


Figure (4.6): The basic idea behind map projections[1]

4.4.1 The Transverse Mercator Projection

The Transverse Mercator projection is a conformal projection. This means that angular measurements made on the projection surface are true. The Projection is based on a cylinder that is slightly smaller than the spheroid and is then flattened out. The method is used by many countries and is especially suited to large countries around the equator.

The Transverse Mercator Projection is defined by:

- False Easting and False Northing.
- Latitude of Origin
- Central Meridian

[1] From Reference No. 12

- Scale on Central meridian
- Zone Width

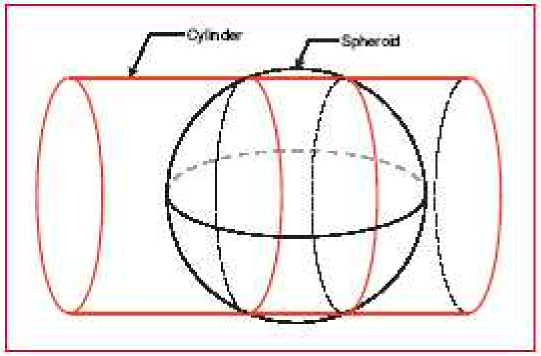


Figure (4.7): Transverse Mercator projection[1]

The **False Easting and False Northing** is defined in order that the origin of the grid projection can be in the lower left hand corner as convention dictates. This does away with the need for negative coordinates.

The **Latitude of Origin** defines the Latitude of the axis of the cylinder. This is normally the equator (in the northern hemisphere). The **Central Meridian** defines the direction of grid north and the longitude of the centre of the projection. Scale varies in an east-west direction. As the cylinder is usually smaller than the spheroid, the **Scale on Central Meridian** is too small, is correct on the ellipses of intersection and is then too large at the edges of the projection. The scale in the north-south direction does not vary. For this reason, the Transverse Mercator projection is most suitable for mapping areas that are long in the north-south direction. The **Zone Width** defines the portion of the spheroid in an east-west direction to which the projection applies.

[1] From Reference No. 12

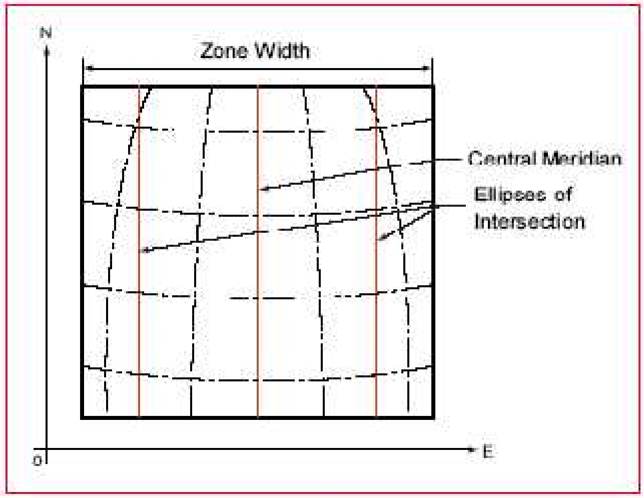


Figure (4.8): Features of the Transverse Mercator projection[1]

Universal Transverse Mercator (UTM)

The UTM projection covers the world between 80ºN and 80ºS latitude. It is a type of Transverse Mercator projection, with many of the defining parameters held fixed. The UTM is split into zones of 6º longitude with adjacent zones overlapping by 30. The one defining parameter is the Central Meridian or Zone Number. (When one is defined, the other is implied).

4.4.2 The Lambert Projection

The Lambert Projection is also a conformal projection based on a cone that intersects the spheroid. It is ideal for small, circular countries, islands and Polar Regions. (See Figure 4.9)

[1] From Reference No. 12

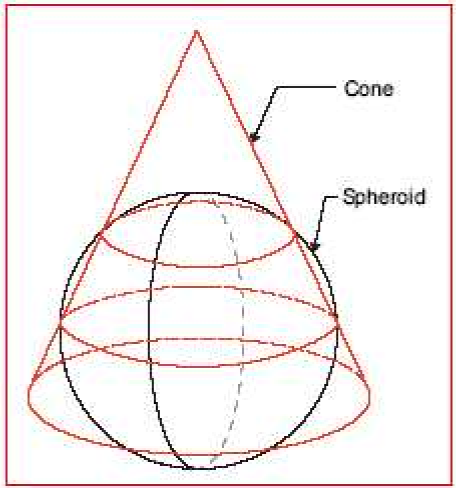


Figure (4.9): The Lambert Projection[1]

The Lambert projection is defined by:

- False Easting and Northing
- Latitude of origin
- Central Meridian
- Latitude of 1st Standard Parallel
- Latitude of 2nd Standard Parallel

The **False Easting and False Northing** are defined in order that the origin of the grid projection can be in the lower left hand corner as convention dictates. This does away with the need for negative coordinates.

The **Latitude of Origin** defines the latitude of the origin of the projection.

The **Central Meridian** defines the direction of grid north and the longitude of the centre of the projection.

[1] From Reference No. 12

The **Latitude of 1st Standard Parallel** defines the latitude at which the cone first cuts the spheroid. This also defines where the influence of scale in the north-south direction is zero.

The **Latitude of 2nd Standard Parallel** defines the second latitude at which the cone cuts the pyramid. The influence of scale will also be zero at this point. The scale is too small between the standard parallels and too large outside them, being defined by the latitudes of the Standard Parallels at which it is zero. Scale in the east-west direction does not vary. (See Figure 4.10)

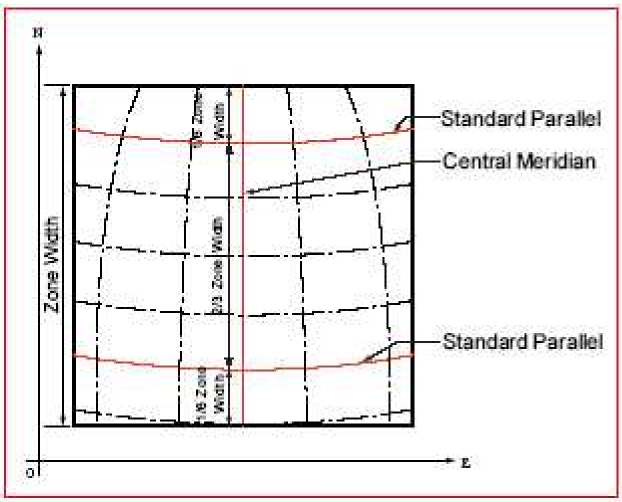


Figure (4.10): Features of the Lambert Projection[1]

[1] From Reference No. 12

4.5 Single Baseline - Multi-Baseline Solutions

A multi-baseline phase data reduction procedure accounts for the inherent correlation between stations observing simultaneously through the functional model for the double-differences containing parameters linking independent baselines. For example, if baselines 1-2 and 2-3 are to be used to generate double-differences, then the coordinate parameters for station 2 appear in the parametric equations for both of the baselines 1-2 and 2-3, hence the baselines are functionally correlated. Further, if the between-station correlations are include in the weight matrix of the observations, the stochastic correlations between the differenced observations are also taken into account.

It must be stressed from the outset however that, in practice, there is often little discernible effect at the few parts per million levels, on GPS solutions, between processing single baselines and processing all session observations in a simultaneous multi-baseline adjustment. However, such a statement must carry several riders:

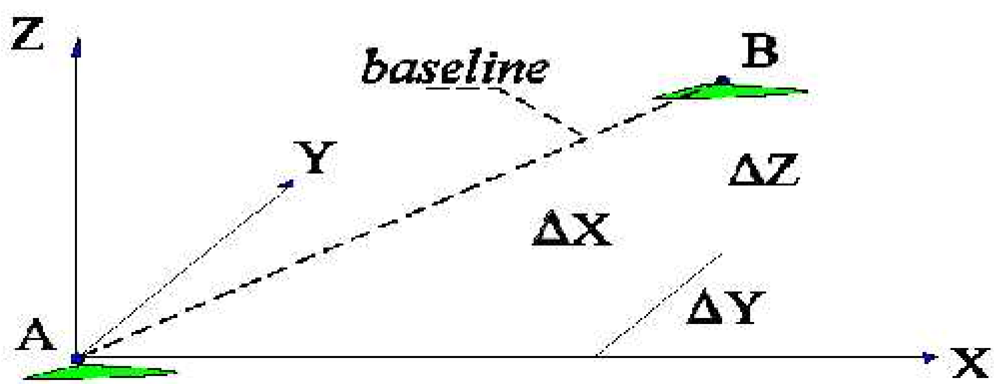
- For example, multi-baseline solutions are theoretically superior with regard to ambiguity resolution for baselines of the order of tens of kilometers, than are single baseline solutions.
- High precision GPS geodesy, particularly for medium and long baselines, demands that all correlations are taken into account.

Baseline is the fundamental and most important line in any geodetic network, and must be established very carefully using the most precise, and accurate methods in measurements all the corrections needed in such a work must be introduced to the observations in this line in order to make minimization to the errors in it.

The baseline was measured by the classical method of trilateration, which is determining a 2D coordinate position for an unknown station by measuring distances from known stations

4.6 GPS Baseline Adjustments

With carrier phase measurements, primary adjustment done by software of a manufacturer to determine vector components of baselines. These observations have a full covariance matrix.



* A and B are points on A and B are builds in PPU in Wadi Al Hareya

Figure (4.11): Baseline sketch

Thus, a network of GPS baselines can be adjusted by the method of least squares to yield the most probable values for the unknown coordinates.

4.7 Reduction Grid to Ground Distance

Any distance calculated from National Grid coordinates will be grid distance. (See Figure 4.11)

If this distance is to be set out on the ground it must:

a. Be divided by the scale factor to give the ellipsoidal distance at mean sea level.

$$S = \frac{G}{F} \quad \ldots\ldots\ldots\ldots\ldots (4.1)$$

Where:

S: Ellipsoidal distance (meter)

G: Grid distance (meter)

F: Scale Factor

b. Have the altitude correction applied to give the horizontal ground distance.

$$C_m = \frac{SH}{R} \quad \ldots\ldots\ldots\ldots\ldots (4.2)$$

Where:

Cm: altitude correction

S: Ellipsoidal distance

R: Radius of the earth

H: Mean height between A and B

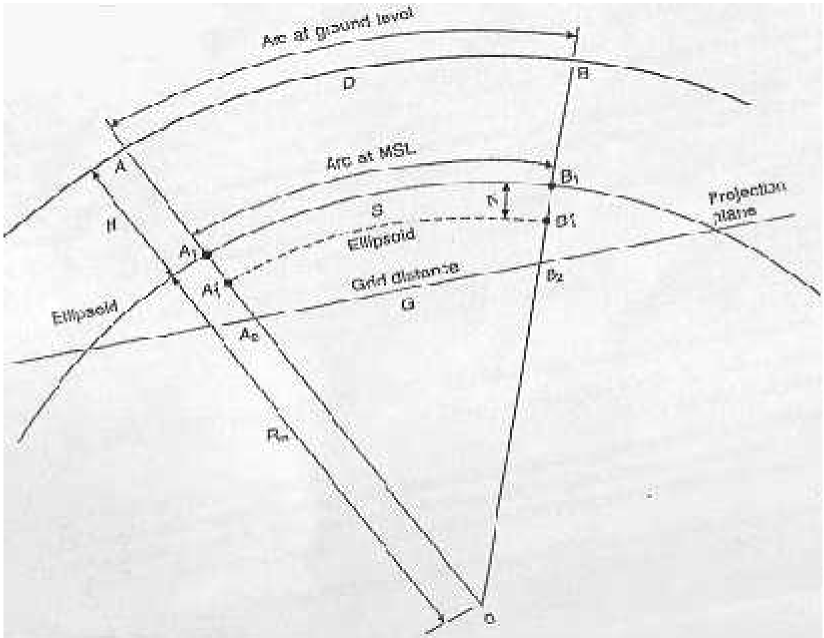


Figure (4.12): Reduction Grid to Ground Distance with Geoid Separation (N)[1]

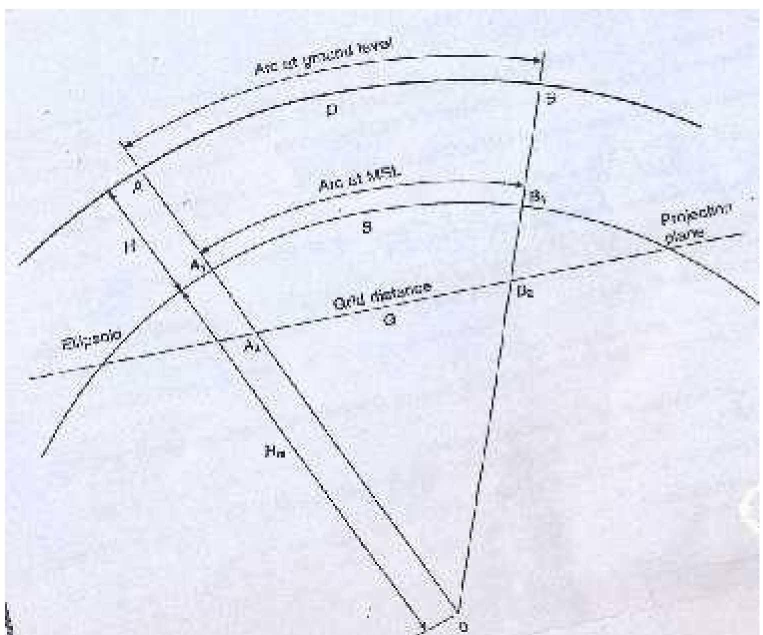


Figure (4.13): Reduction Grid to Ground Distance[2]

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

CHAPTER

5

APPLICATOIN OF LEAST SQUARES

The following contents are going to be covered in this chapter:

CHAPTER FIVE

APPLICATOIN OF LEAST SQUARES

5.1 Background

A Least Squares adjustment involves two models:

The **functional model** relating the measurements and the parameters. The most common approach is to use observation equations of the general form $L = f(x)$**.** To satisfy this relation, actual observations need to be corrected or "adjusted". The linearization of the relation is performed about an approximate set of values for the parameters to be estimated x^0:

$$\begin{aligned}
&L - v = f\,(x')\\
&L - v = f\,(x^0 + \sigma_x)\\
&L - v = f\,(x^0) + A\sigma_x\\
&(L - f\,(x^0)) - v = A\sigma_x
\end{aligned} \qquad \ldots\ldots\ldots (5.1)$$

The expression in brackets is the "observed minus computed" term, or approximate residual, and is denoted by v^0. L is the true and actual observations respectively, **x** and x^0 are the true and approximate parameters respectively. σ_x are the corrections to the approximate parameters and **A** is the design matrix containing the partial derivatives of the observations with respect to the parameters.

The **stochastic model** describing the statistics of the measurement. This is in the form of the weight matrix P, or its inverse the covariance matrix of the observations. For example, all measurements could be independent (that is, a diagonal weight matrix) and have the same standard deviation.

5.2 Application of Least Squares in Processing GPS Data

The least squares method is used at two different stages in processing GPS carrier phase measurements. First, it is applied in the adjustment that yield baseline components between stations from the redundant carrier - phase measurement .Recall that in this procedure, differing techniques are employed to compensate errors in the system and to resolve the cycle ambiguities. In the solution, observation equations are written which contain the differences in coordinate between solutions as parameters. A highly redundant of equations is obtained because, a minimum of four (and often more) satellites are tracked simultaneously using at least two (and often more)) receivers. Furthermore, many repeat observations are taken. This system of equations is solved by least squares to obtain the most probable $\Delta X, \Delta Y, \text{and}\, \Delta Z$ Components of the baseline vectors.

Software designed by manufactures of GPS receivers will process observed phase changes to form the differing observation equations, perform the least square adjustment, and out put the adjusted baseline vector components. The software will also output the covariance matrix, which expresses the correlation between the $\Delta X, \Delta Y, \text{and}\, \Delta Z$ components of each baseline. The software is proprietary and thus cannot be included herein.

The second stage where least squares are employed in processing GPS observations is in adjusting baseline vector components networks. This occurs after the least squares adjustment of carrier-phase observation is completed. In network adjustment, the goal is to make consistent all X coordinates.

The same objective applies for all Y coordinates and for Z coordinates. GPS network shown in figure (5.1).It consists of two control stations and four stations whose coordinates are to be determined. A summary of the baseline measurements obtained from the least squares adjustment of carrier-phase observations. The covariance matrix elements that are listed in the table are used for weighing the observations.

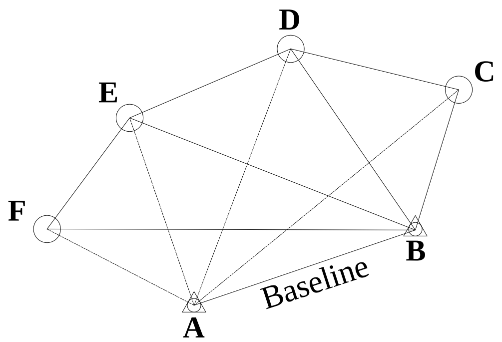


Figure 5.1: GPS survey network

A network adjustment of figure (5.1) should yield adjusted X coordinates for stations (and adjusted coordinate differences between stations) that are all mutually consistent. Specifically for this network, the adjusted X coordinate of station C should be obtained by adding ΔX_{AC} to the X coordinate of station A; and the same value should be obtained by adding ΔX_{BC} to the X coordinate of station B. Equivalent conditions should exist for the Y and Z coordinates.

Observation equation for line *IJ*:

$$
\begin{aligned}
X_C &= X_A + \Delta X_{AC} + V_{X_{AC}} \\
Y_C &= Y_A + \Delta Y_{AC} + V_{Y_{AC}} \qquad \text{……… (5.2)} \\
Z_C &= Z_A + \Delta Z_{AC} + V_{Z_{AC}}
\end{aligned}
$$

$$
\begin{aligned}
X_C &= X_B + \Delta X_{BC} + V_{X_{BC}} \\
Y_C &= Y_B + \Delta Y_{BC} + V_{Y_{BC}} \qquad \text{……… (5.3)} \\
Z_C &= Z_B + \Delta Z_{BC} + V_{Z_{BC}}
\end{aligned}
$$

5.3 Propagation of the GPS Survey

In general, a GPS survey involves the use of a small number of receivers (two or more) to coordinate a large number of stations. The survey must therefore proceed in stages. The minimum GPS phase data reduction is therefore for a single baseline. However the data collected in a single session by all receivers has special characteristics. A GPS session solution can be obtained from a combination of separate baseline solutions, or from one simultaneous session solution. A GPS network solution can be obtained from combining individual session solutions, or in one simultaneous network (multi-session) solution.

GPS processing procedures may therefore be partitioned according to:

- Primary adjustment of the (raw) phase data collected during a session by the deployed GPS receivers. The software generally has the following features:

  - Provided by the receiver manufacturer, in the case of commercial software.
  - Output is the coordinates, and associated VCV matrix, of the stations processed together.
  - The stations may be processed as a single baseline or, more rarely, in the multi-baseline mode.
  - The coordinate results are expressed in a global GPS satellite datum system.

- Secondary adjustment that accepts the output of the primary GPS adjustment as input (that is, as "observations"). Such software has the following features:
  - Not usually developed by the same institution as responsible for the primary GPS software.
  - Can modify and manipulate separate baseline or network solutions in order to combine together a minimally constrained adjustment from separate primary adjustments (from baselines to multi-baseline, from session to multi-session).
  - Can constrain the network adjustment to fit surrounding geodetic control.
  - Accounts for the datum relationships between global and local geodetic datum's.
  - Provides the flexibility to constrain and scale individual GPS adjustments.
  - The results can only be as good as the input primary adjustment data!

5.4 GPS Observation Models

GPS measurements, both pseudo-range and carrier phase, are affected by biases and errors. Different levels of GPS accuracy are associated with a different partitioning of "biases" and "errors". At one extreme, in the case of GPS pseudo-range point positioning, all effects with the exception of the receiver and satellite clock biases are treated as errors. At the other extreme, GPS baseline determination to accuracies of 1 part in 10^8 requires that almost all measurement biases are explicitly accounted for in any solution. In the case of GPS surveying there is a compromise between fully accounting for all biases and keeping the computational burden down by not over-parameter sing the observation model.

* Satellite dependent:

- Ephemeris uncertainties
- Satellite clock uncertainties
- Selective Availability effects

* Receiver dependent:

- Receiver clock uncertainties
- Reference station coordinate uncertainties

* Receiver-Satellite (or observation) dependent:

- Ionospheric delay
- Tropospheric delay
- Carrier phase ambiguity

It is assumed that the data has been cleaned, and that all other effects are indistinguishable from data noise, which the Least Squares adjustment must accommodate.

5.5 GPS Correlation

Correlations express the inter-dependence between variables. For two variables x and y in a linear relationship, the correlation between them is defined as:

$$X_{XY} = \frac{\sigma_{XY}}{\sigma_X \sigma_Y} \quad X_{XY} = \frac{\sigma_{XY}}{\sigma_X \sigma_Y} \quad \text{.........} \ (5.4)$$

where:

σ_x : is the standard deviation of x

σ_y : is the standard deviation of y

σ_{xy} : is the covariance between x and y.

The value of correlation lies in the range -1 and +1. When the correlation of two variables approaches the maximum of ±1, the two variables are said to be highly correlated. It is worth noting that high correlation does not mean that the variations of one are caused by the variations of the others, although it may be the case. In many cases, external influences may be affecting both variables in a similar fashion. There are two types of correlation encountered in GPS measurement and data processing: physical correlation and mathematical correlation.

Physical correlation refers to the correlations between the actual field observations. It arises from the nature of the observations as well as their method of collection. If different observations or sets of observation are affected by common external influences, they are said to be physically correlated. Hence all observations made at the same time at a site may be considered physically correlated because similar atmospheric conditions and clock errors influence the measurements.

Mathematical correlation is related to the parameters in the mathematical model. It can therefore be partitioned into two further classes which correspond to the two components of the mathematical adjustment model:

- Functional Correlation: The physical correlations can be taken into account by introducing appropriate terms into the functional model of the observations. That is, functionally correlated quantities share the same parameter in the observation model. An example is the clock error parameter in the one-way GPS observation model, used to account for the physical correlation introduced into the measurements by the receiver clock and/or satellite clock errors.
- Stochastic Correlation: Stochastic correlation (or statistical correlation) occurs between observations when non-zero off-diagonal elements are present in the variance-covariance (VCV) matrix of the observations. This correlation also appears when functions of the observations are considered (for example, differencing), due to the Law of Propagation of Variances. However, even if the VCV matrix of the observations is diagonal (no stochastic correlation), the VCV matrix of the resultant Least Squares estimates of the parameters will generally be full matrices, and therefore exhibit stochastic correlation.

CHAPTER

6

MEASUREMENTS AND CALCULATIONS

The following contents are going to be covered in this chapter:

CHAPTER SIX

MEASUREMENTS AND CALCULATION

6.1 Introduction

In this chapter we will show the measurements or data which we get in the field surveying. It includes horizontal vertical angles and horizontal distance (gained from total station),and baseline data for the network(covariance matrix element and probable $\Delta X, \Delta Y, \text{and}\, \Delta Z$ components of each baseline captured from GPS).

Then we will calculate the most probable value of the horizontal distance and the height difference between these two control points that were measured using the appropriate equipment using trilateration method.

Also, we calculate errors in total station measurement and geoid separation. Finally, we calculate adjusted coordinates of network, loop of closure, standard deviations for the unknown and standard deviations of an observation.

.

6.2 Coordinates calculations and baseline calibration

All survey connected to the National Grid (NG) should have their measured distance reduced to the horizontal then to the MSL after that multiplied by scale factor and finally, reduced to grid distance.

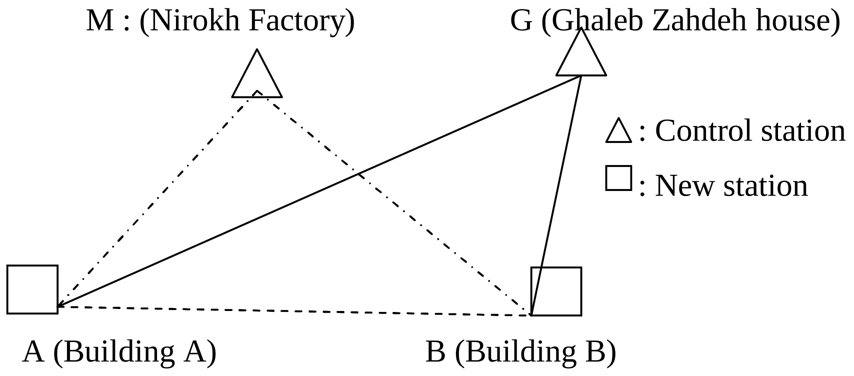


Figure (6.1): Sketch for work

* In the figure (6.1) the point M and G are control pints

* The coordinate of control points are

- Control point (G)

E = 158775.68 m

N = 101189.49 m

Z = 921.57 m

- Control point (M)

E = 158752.46 m

N = 101734.70 m

Z = 899.70 m

Where:

E = X = Easting

N = Y = Northing

Z = Elevation at point

Az = Azimuth angle

* The point (A and B) are unknown of coordinate

Table (6.1): Reading from Instrument (Total Station)

Station	point	Horizontal Distance D (m)	Vertical Distance Δh (m)	Horizontal Angle H . A	Vertical Angle V . A	Prism Height P (m)
G	M			00˚ 00' 00"		
	A	635.486	-5.574	05˚ 14' 18"	90˚ 30' 14"	1.6
		635.486	-5.577	05˚ 14' 12"	90˚ 30' 10"	1.6
		635.486	-5.567	05˚ 14' 13"	90˚ 30' 07"	1.6
		635.486	-5.574	05˚ 14' 11"	90˚ 30' 01"	1.6
G	B	377.523	-5.114	329˚ 04' 19"	90˚ 46' 41"	1.57
		377.523	-5.136	329˚ 04' 08"	90˚ 46' 47"	1.57
		377.522	-5.153	329˚ 04' 10"	90˚ 47' 00"	1.57
		377.523	-5.147	329˚ 04' 04"	90˚ 46' 53"	1.57
M	G			00˚ 00' 00"		
	A	105.133	15.969	213˚ 29' 55"	81˚ 22' 57"	1.6
		105.131	15.967	213˚ 30' 04"	81˚ 22' 52"	1.6
		105.131	15.967	213˚ 29' 56"	81˚ 22' 46"	1.6
		105.132	15.967	213˚ 29' 54"	81˚ 22' 52"	1.6
M	B	294.408	16.353	41˚ 14' 15"	86˚ 49' 11"	1.57
		294.407	16.351	41˚ 14' 18"	86˚ 49' 16"	1.57
		294.406	16.358	41˚ 14' 12"	86˚ 49' 08"	1.57
		294.407	16.354	41˚ 14' 13"	86˚ 49' 09"	1.57
A	G			00˚ 00' 00"		
	B	398.790	0.418	33˚ 58' 05"	89˚ 56' 23"	1.57
		398.791	0.413	33˚ 58' 07"	89˚ 56' 18"	1.57
		398.792	0.427	33˚ 58' 06"	89˚ 56' 19"	1.57
		398.791	0.429	33˚ 58' 03"	89˚ 56' 18"	1.57

Calculation for baseline calibration:

First step:-

We calculate the mean for the Horizontal Distance, the Horizontal Angle, the Vertical Angle and the Vertical Distance using following law:

Arithmetic Mean (M = $\overline{Y}$)

$$M = \overline{Y} = \frac{\sum_{i=1}^{n} Y_i^2}{n}$$

Where

n : is the number of obvervation

- The mean of Horizontal Angle

$$(MGA) = \frac{05^0\ 14'18'' + 05^0\ 14'12'' + 05^0\ 14'13'' + 05^0\ 14'11''}{4} = 05^0\ 14'13.5''$$

MGA = 05° 14' 13.5''

MGB = 329° 04' 10.25''

GMA = 213° 29' 57.25''

GMB = 41° 14' 14.5''

- The mean of Vertical Angle

$$(MGA) = \frac{90^0\ 30'14'' + 90^0\ 30'10'' + 90^0\ 30'07'' + 90^0\ 30'01''}{4} = 90^0\ 30'04''$$

MGA = 90° 30' 04''

MGB = 90° 46' 50.25''

GMA = 81° 22' 51.75''

GMB = 86° 49' 11''

- The mean of Horizontal Distance

$$GA = \frac{635.486 + 635.486 + 635.486 + 635.486}{4} = 635.486$$

GA = 635.486 m

GB = 377.52275 m

MA = 105.13175 m

MB = 294.407 m

AB = 398.79125 m

Second step:-

We calculate standard deviation (S_i) for the Horizontal Distance at 99.7% level confidence using following law:

$$S_i = \pm\sqrt{\frac{\sum_{i=1}^{n}(\overline{Y} - Y)^2}{n-1}}$$

Standard deviation of distance (GA) = $\pm\sqrt{\frac{0}{4-1}} = 0$

$E_{99.7} = 2.756 * 0 = 0$

GB = 0

MA = 0

MB = 0

AB = 0

* No blunder in all reading for any distance.

Coordinates Computations

* Least Squares Solution

Third step:-

Calculate Initial approximate for the unknown station coordinates:

$$E = E_0 + DSin(Az)$$
$$N = N_0 + DCos(Az)$$
$$Az_{(A-B)} = \tan^{-1}\frac{E_B - E_A}{N_B - N_A}$$

* Instrument on control point (G)

$$Az_{(G-M)} = \tan^{-1}\frac{E_M - E_G}{N_M - N_G} = \tan^{-1}\frac{158752.46 - 158775.68}{101734.70 - 101189.49} = -02^0\ 26'\,19''$$
$$Az_{(G-M)} = 360 - 02^0\ 26'\,19'' = 357^0\ 33'\ 41''$$
$$Az_{(G-A)} = Az_{(G-M)} + \text{Horizantal Angle (MGA)}$$
$$Az_{(G-A)} = 357^0\ 33'\ 41'' + 05^0\ 14'\,13.5'' - 360 = 02^0\ 47'\ 54.5''$$
$$Az_{(G-B)} = Az_{(G-M)} + \text{Horizantal Angle (MGB)}$$
$$Az_{(G-B)} = 357^0\ 33'\ 41'' + 329^0\ 04'\,10.25'' - 360 = 326^0\ 37'\ 51.2''$$

$$E_A = 158775.68 + 635.486\ \text{Sin}\ 02^0\ 47'\ 54.5'' = 158806.706$$
$$N_A = 101189.49 + 635.486\ \text{Cos}\ 02^0\ 47'\ 54.5'' = 101824.218$$
$$E_B = 158775.68 + 377.52275\ \text{Sin}\ 326^0\ 37'\ 51.2'' = 158568.031$$
$$N_B = 101189.49 + 377.52275\ \text{Cos}\ 326^0\ 37'\ 51.2'' = 101504.776$$

* Instrument on control point (M)

$$Az_{(M-G)} = \tan^{-1}\frac{E_G - E_M}{N_G - N_M} = \tan^{-1}\frac{158775.68 - 158752.46}{101189.49 - 101734.70} = -02^0\ 26'19''$$

$$Az_{(M-G)} = 180 - 02^0\ 26'19'' = 177^0\ 33'41''$$

$$Az_{(M-A)} = Az_{(M-G)} + \text{Horizantal Angle (GMA)}$$

$$Az_{(M-A)} = 177^0\ 33'41'' + 213^0\ 29'57.25'' - 360 = 31^0\ 03'\ 38.25''$$

$$Az_{(M-B)} = Az_{(M-G)} + \text{Horizantal Angle (GMB)}$$

$$Az_{(M-B)} = 177^0\ 33'41'' + 41^0\ 14'\ 14.5'' = 218^0\ 47'\ 55.5''$$

$$E_A = 158752.46 + 105.13175\ \text{Sin}\ 31^0\ 03'\ 38.25'' = 158806.702$$

$$N_A = 101734.70 + 105.13175\ \text{Cos}\ 31^0\ 03'\ 38.25'' = 101824.758$$

$$E_B = 158752.46 + 294.407\ \text{Sin}\ 218^0\ 47'\ 55.5'' = 158567.988$$

$$N_B = 101734.70 + 294.407\ \text{Cos}\ 218^0\ 47'\ 55.5'' = 101505.253$$

* The mean of Initial 2D-Coordinat

$$E_A = \frac{158806.702 + 158806.706}{2} = 158806.704$$

$$N_A = \frac{101824.248 + 101824.758}{2} = 101824.488$$

$$E_B = \frac{158568.031 + 158567.988}{2} = 158568.009$$

$$N_B = \frac{101505.253 + 101504.776}{2} = 101505.014$$

Fourth step:-

We calculate the heights of points (A, B) using the following law:

$$Z = i + \text{elevation at point} + D\tan\sim \text{ - heigh Target(prism)}$$

Where :

Z : Elevation at new point

D : Horizantal Distance

i : Instrument heigh

$\sim = 90^0 -$ vertical angle reading

$D\tan\sim =$ vertial distance

* Calculate the elevation (A, B) from point (G)

Instrument height = 1.57 m

Prism height on point (A) = 1.6 m

Prism height on point (B) = 1.57 m

Elevation at point (G) = 921.27 m

Vertical distance at (A) = -5.573 m

Vertical distance at (B) = -5.1375 m

- Elevation at point (A) = 921.57 + 1.57 - 5.573 - 1.6 = 915.667 m
- Elevation at point (B) = 921.27 + 1.57 - 5.1375 - 1.57 = 916.1325 m

For check

* Calculate the elevation (A, B) from point (M)

Instrument height = 1.59 m

Prism height on point (A) = 1.6 m

Prism height on point (B) = 1.57 m

Elevation at point (G) = 899.7 m

Vertical distance at (A) = 15.9675 m

Vertical distance at (B) = 16.354 m

- Elevation at point (A) = 899.70 + 1.59 + 15.9675 - 1.6 = 915.6575 m
- Elevation at point (B) = 899.70 + 1.59 + 16.354 - 1.57 = 916.074 m

Fifth step:-

Formulate (X) and (K) matrices:

The elements of the (X) matrix consist of the ∂x and ∂y (called correction). The value of (K) matrix is derived by subtracting computed distance, based on the initial coordinate so the initial value denoted by $(\text{quantity})_0$.

$$K = \begin{bmatrix} GA-(GA)_0 \\ GB-(GB)_0 \\ MA-(MA)_0 \\ MB-(MB)_0 \\ AB-(AB)_0 \end{bmatrix} \quad , \quad X = \begin{bmatrix} \partial x_A \\ \partial y_A \\ \partial x_B \\ \partial y_B \end{bmatrix}$$

* Calculate the Jacobin matrix. The J matrix is found using proto type equation for distance.

Distance observation equation from figure (6.2):

$$\frac{\left(X_I - X_j\right)_0}{(ij)_0}\partial x_i + \frac{\left(Y_I - Y_j\right)_0}{(ij)_0}\partial y_i + \frac{\left(X_j - X_i\right)_0}{(ij)_0}\partial x_j + \frac{\left(Y_J - Y_I\right)_0}{(ij)_0}\partial y_j = K_{ij} + V_{ij}$$

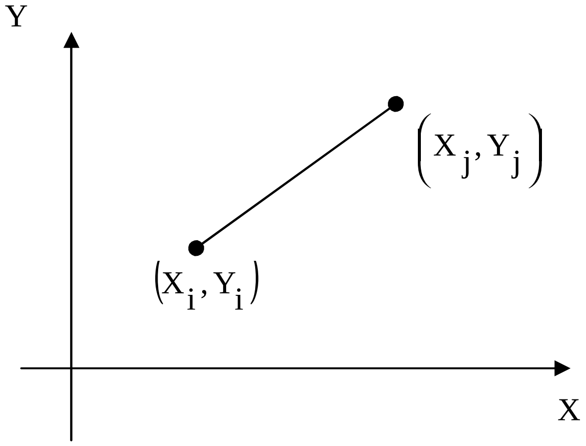


Figure (6.2): Distance observation equation

$$GA = \sqrt{(X_A - X_G)^2 + (Y_A - Y_G)^2} \text{..........(1)}$$

$$GB = \sqrt{(X_B - X_G)^2 + (Y_B - Y_G)^2} \text{...........(2)}$$

$$MA = \sqrt{(X_A - X_M)^2 + (Y_A - Y_M)^2} \text{..........(3)}$$

$$MB = \sqrt{(X_B - X_M)^2 + (Y_B - Y_M)^2} \text{..........(4)}$$

$$AB = \sqrt{(X_A - X_B)^2 + (Y_A - Y_B)^2} \text{..........(5)}$$

$$\frac{\partial_{GA}}{\partial x_A} = \frac{1*2\left(X_A - X_G\right)}{2\sqrt{\Delta X^2 + \Delta Y^2}} = \frac{X_A - X_G}{\left(AG\right)_0} ....(6)$$

$$\frac{\partial_{GA}}{\partial y_A} = \frac{1*2\left(Y_A - Y_G\right)}{2\sqrt{\Delta X^2 + \Delta Y^2}} = \frac{Y_A - Y_G}{\left(AG\right)_0} ....(7)$$

$$\frac{\partial_{GA}}{\partial x_B} = 0..........(8)$$

$$\frac{\partial_{GA}}{\partial y_B} = 0..........(9)$$

$$\vdots$$

$$\vdots$$

$$\vdots$$

$$\frac{\partial_{AB}}{\partial x_A} = \frac{1*2\left(X_A - X_B\right)}{2\sqrt{\Delta X^2 + \Delta Y^2}} = \frac{X_A - X_B}{\left(AB\right)_0} ....(24)$$

$$\frac{\partial_{AB}}{\partial y_A} = \frac{1*2\left(Y_A - Y\ \right)}{2\sqrt{\Delta X^2 + \Delta Y^2}} = \frac{Y_A - Y_B}{\left(AB\right)_0} ....(25)$$

$$\frac{\partial_{AB}}{\partial x_B} = \frac{1*-2\left(X_A - X_B\right)}{2\sqrt{\Delta X^2 + \Delta Y^2}} = \frac{X_B - X_A}{\left(AB\right)_0} ....(26)$$

$$\frac{\partial_{AB}}{\partial y_B} = \frac{1*-2\left(Y_A - Y_B\right)}{2\sqrt{\Delta X^2 + \Delta Y^2}} = \frac{Y_B - Y_A}{\left(AB\right)_0} ....(27)$$

$$J = \begin{bmatrix} \frac{\partial_{GA}}{\partial x_A} & \frac{\partial_{GA}}{\partial y_A} & \frac{\partial_{GA}}{\partial x_B} & \frac{\partial_{GA}}{\partial y_B} \\ \frac{\partial_{GB}}{\partial x_A} & \frac{\partial_{GB}}{\partial y_A} & \frac{\partial_{GB}}{\partial x_B} & \frac{\partial_{GA}}{\partial y_B} \\ \frac{\partial_{MA}}{\partial x_A} & \frac{\partial_{MA}}{\partial y_A} & \frac{\partial_{MA}}{\partial x_B} & \frac{\partial_{MA}}{\partial y_B} \\ \frac{\partial_{MB}}{\partial x_A} & \frac{\partial_{MB}}{\partial y_A} & \frac{\partial_{MB}}{\partial x_B} & \frac{\partial_{MB}}{\partial y_B} \\ \frac{\partial_{AB}}{\partial x_A} & \frac{\partial_{AB}}{\partial y_A} & \frac{\partial_{AB}}{\partial x_B} & \frac{\partial_{BA}}{\partial y_B} \end{bmatrix}$$

$$J = \begin{bmatrix} \frac{(X_A)_0 - X_G}{(GA)_0} & \frac{(Y_A)_0 - Y_G}{(GA)_0} & 0 & 0 \\ 0 & 0 & \frac{(X_B)_0 - X_G}{(GB)_0} & \frac{(Y_B)_0 - Y_G}{(GB)_0} \\ \frac{(X_A)_0 - X_M}{(MA)_0} & \frac{(Y_A)_0 - Y_M}{(MA)_0} & 0 & 0 \\ 0 & 0 & \frac{(X_B)_0 - X_M}{(MB)_0} & \frac{(Y_B)_0 - Y_M}{(MB)_0} \\ \frac{(X_A)_0 - (X_B)_0}{(AB)_0} & \frac{(Y_A)_0 - (Y_B)_0}{(AB)_0} & \frac{(X_B)_0 - (X_A)_0}{(BA)_0} & \frac{(Y_B)_0 - (Y_A)_0}{(BA)_0} \end{bmatrix}$$

$$K = \begin{bmatrix} GA - (GA)_0 \\ GB - (GB)_0 \\ MA - (MA)_0 \\ MB - (MB)_0 \\ AB - (AB)_0 \end{bmatrix} , \quad X = \begin{bmatrix} \partial x_A \\ \partial y_A \\ \partial x_B \\ \partial y_B \end{bmatrix} , \quad V = \begin{bmatrix} V_{GA} \\ V_{GB} \\ V_{MA} \\ V_{MB} \\ V_{AB} \end{bmatrix}$$

$$J^T = \begin{bmatrix} \frac{(X_A)_0 - X_G}{(GA)_0} & 0 & \frac{(X_A)_0 - X_M}{(MA)_0} & 0 & \frac{(X_A)_0 - (X_B)_0}{(AB)_0} \\ \frac{(Y_A)_0 - Y_G}{(GA)_0} & 0 & \frac{(Y_A)_0 - Y_M}{(MA)_0} & 0 & \frac{(Y_A)_0 - (Y_B)_0}{(AB)_0} \\ 0 & \frac{(X_B)_0 - X_G}{(GB)_0} & 0 & \frac{(X_B)_0 - X_M}{(MB)_0} & \frac{(X_B)_0 - (X_A)_0}{(BA)_0} \\ 0 & \frac{(Y_B)_0 - Y_G}{(GB)_0} & 0 & \frac{(Y_B)_0 - Y_M}{(MB)_0} & \frac{(Y_B)_0 - (Y_A)_0}{(BA)_0} \end{bmatrix}$$

* Formulate the weight (W) matrix:

$$\mathrm{W} = \begin{bmatrix} \frac{1}{\mathrm{S}^2} & 0 & 0 & 0 & 0 \\ 0 & \frac{1}{\mathrm{S}^2} & 0 & 0 & 0 \\ 0 & 0 & \frac{1}{\mathrm{S}^2} & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{\mathrm{S}^2} & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{\mathrm{S}^2} \end{bmatrix} = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

* Solve the matrix system. The problem is iterative and need more than one iteration to reach minimum corrections.

$$\mathrm{J} * \mathrm{X} = \mathrm{K} + \mathrm{V}$$
$$\mathrm{J} * \mathrm{X} = \mathrm{K}$$
$$\mathrm{X} = \left(\mathrm{J}^{\mathrm{T}} * \mathrm{W} * \mathrm{J}\right)^{-1} * \left(\mathrm{J}^{\mathrm{T}} * \mathrm{W} * \mathrm{K}\right)$$
$$\mathrm{X} = \mathrm{N}^{-1} * \left(\mathrm{J}^{\mathrm{T}} * \mathrm{W} * \mathrm{K}\right)$$

* We calculate the Initial distance by using the following law:

$$AB = \sqrt{(X_A - X_B)^2 + (Y_A - Y_B)^2}$$

$$(GA)_0 = \sqrt{(158775.68 - 158806.704)^2 + (101189.49 - 101824.488)^2}$$
$$= 635.755 \text{ m}$$

$$(GB)_0 = 377.734 \text{ m}$$
$$(MA)_0 = 104.901 \text{ m}$$
$$(MB)_0 = 294.579 \text{ m}$$
$$(AB)_0 = 398.796 \text{ m}$$

* Jacobean (J) Matrix and K,X matrix:

$$J = \begin{bmatrix} 0.049 & 0.999 & 0 & 0 \\ 0 & 0 & -0.549 & 0.835 \\ 0.517 & 0.856 & 0 & 0 \\ 0 & 0 & -0.626 & -0.779 \\ 0.598 & 0.801 & -0.598 & -0.801 \end{bmatrix}, K = \begin{bmatrix} -0.269 \\ -0.211 \\ 0.231 \\ -0.172 \\ -0.005 \end{bmatrix}, \quad X = \begin{bmatrix} \partial x_A \\ \partial y_A \\ \partial x_B \\ \partial y_B \end{bmatrix}$$

$$J^T = \begin{bmatrix} 0.049 & 0 & 0.517 & 0 & 0.598 \\ 0.999 & 0 & 0.856 & 0 & 0.801 \\ 0 & -0.549 & 0 & -0.626 & -0.598 \\ 0 & 0.835 & 0 & -0.779 & -0.801 \end{bmatrix}$$

Inverse Matrix (N)

$$N^{-1} = \begin{bmatrix} 5.303057 & -1.89321 & 0.700689 & 0.49821 \\ -1.89321 & 1.162469 & -0.08511 & -0.06052 \\ 0.700689 & -0.08511 & 1.237737 & -0.17888 \\ 0.49821 & -0.06052 & -0.17888 & 0.663386 \end{bmatrix}$$

$$X = \begin{bmatrix} \partial x_A \\ \partial y_A \\ \partial x_B \\ \partial y_B \end{bmatrix} = \begin{bmatrix} 0.829243 \\ -0.29964 \\ 0.365914 \\ -0.00987 \end{bmatrix}$$

* After final iteration adjusted coordinates are computed, also residuals

Table 6.2: adjusted coordinates

STATION	ADJUSTED X	ADJUSTED Y
A	158807.533	101824.189
B	158568.374	101505.006

Sixth step:-

Grid Distance = $\sqrt{\Delta E^2 + \Delta N^2}$

Grid Distance = $\sqrt{(158807.533 - 158568.374)^2 + (101824.189 - 101505.006)^2}$

Grid Distance = 398.842 m

* calculate ellipsoidal distance at mean sea level by using following low:

$$S = \frac{G}{F}$$

Where:

S: Ellipsoidal distance (meter)

G: Grid distance (meter)

F: Scale Factor

$$F = F_0 + K(\Delta E)^2$$

Where:

F: Scale Factor at any point

F_0: Scale Factor at central meridian = 1.000000

K: constant value = $1.228*10^{-14}$

ΔE: Is the difference in easting between central meridians and required point

ΔE = (E – False Easting)

Note:

* False Easting in Palestine = 170251.555 m

* Calculate E_{mid}

$$E_{mid} = \frac{E_A + E_B}{2} = 158687.954 \text{ m}$$

$$F_{mid} = 1.00 + 1.228*10^{-14}(158687.954 - 170251.555)^2 = 1.000001642$$

$$F_A = 1.00 + 1.228*10^{-14}(158807.53 - 170251.555)^2 = 1.000001608$$

$$F_{mid} = 1.00 + 1.228*10^{-14}(158568.37 - 170251.555)^2 = 1.000001676$$

$$F_{Total} = \frac{F_A + F_B + F_{mid}}{3} = 1.000001642$$

$$S = \frac{398.842}{1.000001642} = 398.8413 \text{ m}$$

* Calculate altitude correction by following low:

$$C_m = \frac{SH}{R} = \frac{398.8413 * 915.89975}{6384100} = +0.05722 \text{ m}$$

Where:

S: Ellipsoidal distance

R: Radius of the earth

H: Mean height between A and B

* Horizontal distance of ground level = S + C

= 398.8413 + 0.05722 = 398.8985 m

***Atmospheric correction factor and Humidity**

- The SET is designed so that the correction factor is 0 ppm at a temperature of 15˚C (59˚F), an atmospheric pressure of 1013hPa (29.9inchHg) and humidity of 0%.
- By inputting the temperature and air pressure values, the correction value is calculated and set into the memory. PPM value can be input directly as well.

- The formula used is as follows:

$$\text{ppm} = 282.59 - \frac{0.2942 * \text{P (hPa)}}{1 + 0.003661 * \text{T}(^0\text{C})}$$

* At 26˚C and 738 mmHg (984.375hPa) from (Thermometer and Parometer) ppm is:

$$ppm = 282.59 - \frac{0.2942 * 984.375}{1 + 0.003661 * 26} = 18.15 \approx 18$$

*** Errors in Electronic Distance Measurements**

$$\sigma_D = \pm\sqrt{\sigma_i^2 + \sigma_t^2 + a^2 + \left(D * b_{ppm}\right)^2}$$

Where:

σ_D : Is the Error in the Measured Distance (D)

σ_i : Instrument Centering Error

σ_t : Target Centering Error

a and b : Are the Instrument's Specifred Accuracy Parameters

D : Distance Between point A and B

$$\sigma_D = \pm\sqrt{(0.003)^2 + (0.003)^2 + (0.003)^2 + \left(\frac{0.002}{1*10^6} * 398.49\right)^2} = \pm 0.005196\, m$$

*** Geoid separation**

Geoid separation computed by using following low:

$$h = N + H$$

Where:
h: Ellipsoidal height(m)
H: Orthometric height (m)
N: Geoid separation

* From using GPS. Ellipsoidal height was determined using GPS static technique

A = 931.595
B = 932.017

* Orthometric height was calculated using grid coordinate

A = 915.667
B = 916.1325

$$N_a = h_a - H_a = 931.595 - 915.667 = 15.928 \text{ m}$$

$$N_b = h_b - H_b = 932.017 - 916.1325 = 15.884 \text{ m}$$

$$\text{Average Geoid separation} = \frac{N_a + N_b}{2} = 15.906 \text{ m}$$

6.3 Adjustment of GPS Networks

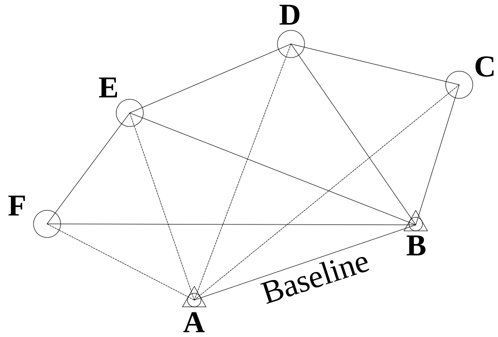


Figure 6.3: GPS survey network

In this figure, stations A and B are control stations, and stations C, D, E, and F are points of unknown position.

A: Point above Building A in Wadi Al-Hareya

B: Point above Building B in Wadi Al-Hareya

C: Point above Building Abu Rumman (Friends of Fawzikawah IT Center of Excellence)

D: Point above Building Al-Shareya school

E: Point above Building A in Abu Ktelah

F: Point above Building Ein Sara (PPU)

During set up GPS instrument we obtained measurements which organize in the tables (6.3, and 6.4).

We obtained Palestinian coordinates by using parameters from appendix B page 132, and entering them to the (GPS receiver 5700) before using GPS technique static, to transform reading coordinates from WGS84 to Palestinian Grid coordinates.

&6.٣جدول

6.4

* First step:

Calculate loop of closure:

GPS networks will typically consist of many interconnected closed loops. In the network of figure (6.3), a closed loop is formed by points ACBFA. Similarly, ADBFA, BCADB, and so on, are other closed loops. For each closed loop, the algebraic sum of ΔX Components should equal zero. The same condition should exist for ΔY and ΔZ Components. An unusually large Closure within any loop will indicate that either a blunder or large error exists in one (or more) of the baselines of the loop.

$$\mathrm{CX} = \sum \mathrm{X's}$$
$$\mathrm{CY} = \sum \mathrm{Y's}$$
$$\mathrm{CZ} = \sum \mathrm{Z's}$$

Check X in loop: A-C-B-F-A

$$CX = \Delta X_{AC} + \Delta X_{CB} + \Delta X_{BF} + \Delta X_{FA}$$
$$CY = \Delta Y_{AC} + \Delta Y_{CB} + \Delta Y_{BF} + \Delta Y_{FA}$$
$$CZ = \Delta Z_{AC} + \Delta Z_{CB} + \Delta Z_{BF} + \Delta Z_{FA}$$

CX = -428.464 - -189.3060 + 751.6391- 512.480 = 0.0011 m

CY = 1415.792 - 1734.9751 + 3113.9691- 2794.787 = - 0.0010 m

CZ = 50.832 - 50.419 - -3.713 + 3.301 = 0.0010 m

Loop closure error, *ce*, is:

$$\mathrm{ce} = \sqrt{\mathrm{cx}^2 + \mathrm{cy}^2 + \mathrm{cz}^2}$$

$$\mathrm{ce} = \sqrt{(0.0011)^2 + (-0.001)^2 + (0.001)^2} = 1.7916 * 10^{-3}$$

and ppm is:

$$\frac{ce}{\text{Loop length}} * 1{,}000{,}000$$

$$\text{Loop length} = \text{dis.AC} + \text{dis.CB} + \text{dis.BF} + \text{dis.FA}$$

$$\text{dis.JI} = \sqrt{\Delta X_{JI}^2 + \Delta Y_{JI}^2 + \Delta Z_{JI}^2}$$

$$\text{dis.AC} = \sqrt{(-428.464)^2 + (1415.792)^2 + (50.832)^2} = 1480.0747 \text{ m}$$

$$\text{dis.CB} = \sqrt{(189.306)^2 + (-1734.9751)^2 + (50.419)^2} = 1746.004 \text{ m}$$

$$\text{dis.BF} = \sqrt{(751.6391)^2 + (3113.9691)^2 + (-3.713)^2} = 3203.4011 \text{ m}$$

$$\text{dis.FA} = \sqrt{(512.4800)^2 + (2794.787)^2 + (3.301)^2} = 2841.3871 \text{ m}$$

$$\text{Loop length} = 9270.8633 \text{ m}$$

$$\frac{ce}{\text{Loop length}} * 1{,}000{,}000 = \frac{1.7916 * 10^{-3}}{9270.8633} * 1{,}000{,}000 = 0.19325 \text{ ppm}$$

Second step:

Analysis of check baseline

Used to check the accuracy of the GPS measurement system, and the control being used:

Table 6.5 Measured control point data for the network

Control Point			
Name Point	**Northing (m)**	**Easting (m)**	**Elevation (m)**
A	101824.174	158807.513	915.653
B	101505.089	158568.462	916.048

Table 6.6 Fixed control point data for the network

Control Point			
Name Point	**Northing (m)**	**Easting (m)**	**Elevation (m)**
A	101824.189	158807.533	915.662
B	101505.006	158568.374	916.074

Table 6.7 Comparisons of measured and fixed baseline components

Component (1)	Measured (m) (2)	Fixed (m) (3)	Difference (4)	PPM * (5)
ΔX	239.051	239.159	0.108	270.7
ΔY	319.085	319.183	0.103	258.24
ΔZ	0.395	0.413	0.017	42.62

* The total baseline length used in computing these ppm values was 398.842 m, which was derived from square root of the sum of the squares of ΔX, ΔY, and ΔZ values.

1. List the vector component of the baseline.
2. List the GPS measured value for the vector component.
3. List the vector component as derived from the control station coordinates.
4. Absolute value of the difference of the values listed in column (2) and (3).
5. The PPM (column 4) is computed as:

$$ppm = \frac{\text{difference}}{\sqrt{\Delta X^2 + \Delta Y^2 + \Delta Z^2}} * 1{,}000{,}000$$

* Third step

Least squares adjustment of GPS networks

As noted earlier, because GPS networks redundant measurements, they must be adjusted to make all coordinate differences consistent. In applying least squares to the problem of adjusting baselines in GPS network, observation equations are written that relate station coordinates to the observed coordinate differences and their residual error. To illustrate the procedure, consider of figure (6.3).For line AC of this figure, an observation equation can be written for each measured baseline component as:

$$X_C = X_A + \Delta X_{AC} + V_{X_{AC}}$$
$$Y_C = Y_A + \Delta Y_{AC} + V_{Y_{AC}}$$
$$Z_C = Z_A + \Delta Z_{AC} + V_{Z_{AC}}$$

Similarly, the observation equations for the base line components of line AD are

$$X_D = X_A + \Delta X_{AD} + V_{X_{AD}}$$
$$Y_D = Y_A + \Delta Y_{AD} + V_{Y_{AD}}$$
$$Z_D = Z_A + \Delta Z_{AD} + V_{Z_{AD}}$$

Observation equations of the form above would be written for all measured baseline in any figure. For figure (6.3) there were a total of 8 measured baselines, so the number of observation equations that can be developed is 24.Also, each of stations C,D,E, and F has three unknown coordinates, for a total of 12 unknowns in the network. Thus there are 24-12=12 redundant observations in the network. The 24 observation equations can be expressed in matrix form as:

AX=L+V

If the observation equations for adjusting the network of figure (6.3) are written in the same order that the measurements are listed in tables (6.3 & 6.4), the A, X, and V matrices would be.

Coefficient matrix similar to that in differential leveling, however, weight matrix has off-diagonal elements since baseline measurements are indirect observations.

- Formulate (A) matrix

The elements of the (A) matrix are coefficients of observation equations:

$$A = \begin{bmatrix}
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \\
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1
\end{bmatrix}$$

* Formulate (X), (L), and (V)

X matrix: is the unknowns

L matrix: is the observation

V matrix: are the residuals

$$X = \begin{bmatrix} X_C \\ Y_C \\ Z_C \\ X_D \\ Y_D \\ Z_D \\ X_E \\ Y_E \\ Z_E \\ X_F \\ Y_F \\ Z_F \end{bmatrix} \qquad L = \begin{bmatrix} X_A + \Delta X_{AC} \\ Y_A + \Delta Y_{AC} \\ Z_A + \Delta Z_{AC} \\ X_A + \Delta X_{AD} \\ Y_A + \Delta Y_{AD} \\ Z_A + \Delta Z_{AD} \\ X_A + \Delta X_{AE} \\ Y_A + \Delta Y_{AE} \\ Z_A + \Delta Z_{AE} \\ X_A + \Delta X_{AF} \\ Y_A + \Delta Y_{AF} \\ Z_A + \Delta Z_{AF} \\ \vdots \\ X_B + \Delta X_{BF} \\ Y_B + \Delta Y_{BF} \\ Z_B + \Delta Z_{BF} \end{bmatrix} \qquad V = \begin{bmatrix} V_{X_{AC}} \\ V_{Y_{AC}} \\ V_{Z_{AC}} \\ V_{X_{AD}} \\ V_{Y_{AD}} \\ V_{Z_{AD}} \\ V_{X_{AE}} \\ V_{Y_{AE}} \\ V_{Z_{AE}} \\ V_{X_{AF}} \\ V_{Y_{AF}} \\ V_{Z_{AF}} \\ \vdots \\ V_{X_{BF}} \\ V_{Y_{BF}} \\ V_{Z_{BF}} \end{bmatrix}$$

w مصفوفة بالرموز+ ارقام

w بالارقام مصفوفات

$$L = \begin{bmatrix} 158379.069 \\ 103239.981 \\ 966.494 \\ 158418.978 \\ 103710.143 \\ 955.945 \\ 158292.902 \\ 106237.34 \\ 949.092 \\ 159320.013 \\ 104618.976 \\ 912.361 \\ 158379.068 \\ 103239.9811 \\ 966.493 \\ 158418.9778 \\ 103710.1431 \\ 955.944 \\ 158292.9021 \\ 106237.338 \\ 949.091 \\ 159320.0131 \\ 104618.975 \\ 912.361 \end{bmatrix}$$

* Solve the matrix system by using following law

$$X = \left(A^T W\, A\right)^{-1} * \left(A^T W\, L\right)$$

$$X = N^{-1} * \left(A^T W\, L\right)$$

$$X = \begin{bmatrix} X_C \\ Y_C \\ Z_C \\ X_D \\ Y_D \\ Z_D \\ X_E \\ Y_E \\ Z_E \\ X_F \\ Y_F \\ Z_F \end{bmatrix} = \begin{bmatrix} 158379.068\,6 \\ 103239.980\,9 \\ 966.493733\,3 \\ 158418.977\,9 \\ 103710.143\,1 \\ 955.944576\,4 \\ 158292.902\,1 \\ 106237.338\,6 \\ 949.091374\,1 \\ 159320.013\,2 \\ 104618.975\,9 \\ 912.360866\,3 \end{bmatrix}$$

* Calculate the residuals matrix

$$V = A * X - L$$

$$V = \begin{bmatrix} -0.0004426 \\ -0.0001459 \\ -0.0002667 \\ -8.442*10^{-5} \\ 5.6095*10^{-5} \\ -0.0004236 \\ 0.00013733 \\ -0.0013717 \\ -0.0006259 \\ 0.00018296 \\ -0.000146 \\ -0.0001337 \\ 0.00055742 \\ -0.0002459 \\ -0.0001337 \\ 0.00011558 \\ -4.391*10^{-5} \\ 0.00057642 \\ 3.7334*10^{-5} \\ 0.00062828 \\ 0.00037415 \\ 8.2956*10^{-5} \\ 0.00085396 \\ 0.00086633 \end{bmatrix}$$

Fourth step:-

*** Compute reference standard deviation**

The standard deviation of unit weight for a weighted set of observations is:

$$S_0 = \sqrt{\frac{V^T W V}{m - n}}$$

where :

m : is the number of observations

n : is the number of unknowns

$$V^T W V = 3.3203543833$$

$$S^2 = \frac{V^T W V}{m - n} = \frac{3.320354383}{12} = 0.276696199$$

$$S_0 = \sqrt{S^2} = 0.5260192$$

*** Compute standard deviation for the unknowns**

$$S_i = S_0 \sqrt{Q}$$

$$S_0 = 0.526$$

$$Q = N^{-1} = \left(A^T W A\right)^{-1}$$

$$S_{X_C} = 0.526\sqrt{1.386 * 10^{-6}} = 6.192 * 10^{-4}$$

$$S_{Y_C} = 0.526\sqrt{6.94 * 10^{-7}} = 4.381 * 10^{-4}$$

$$S_{Z_C} = 0.526\sqrt{5.412 * 10^{-6}} = 3.869 * 10^{-4}$$

$$\vdots$$

and so on

Table 6.8 Adjusted Coordinates

Station	X**(Easting)**	Y**(Northing)**	Z	Sx	Sy	Sz
A	158807.533	101824.189	915.662			
B	158568.374	101505.006	916.074			
C	158379.0686	103239.9809	966.493	$6.192*10^{-4}$	$4.381*10^{-4}$	$3.869*10^{-4}$
D	158418.9779	103710.1431	955.944	$6.320*10^{-4}$	$4.524*10^{-4}$	$5.114*10^{-4}$
E	158292.9021	106237.3386	949.091	$7.033*10^{-4}$	$6.516*10^{-4}$	$6.536*10^{-4}$
F	159320.0132	104618.9759	912.360	$6.290*10^{-4}$	$5.863*10^{-4}$	$4.863*10^{-4}$

Fifth step:-

Computed estimated standard deviation of an observation and slope distance:

$$S_{LL}^2 = S_0^2 AQA^T$$

$$S_{AC} = S_0 * \sqrt{AQA^T}$$

$$S_{AC} = 0.526\sqrt{1.38*10^{\ 6} + 6.94*10^{\ 7} + 5.41*10^{\ 7}} = 8.5*10^{\ 4}$$

Computed slope distance

$$AC = \sqrt{\Delta X^2 + \Delta Y^2 + \Delta Z^2} = 1480.078 \text{ m}$$

Table 6.9 Distance Vectors and Residuals

From	**To**	$\pm S$	Slope Distance **(m)**	V_X	V_Y	V_Z
A	C	$8.50*10^{-4}$	1480.078	-0.0004426	-0.0001459	-0.0002667
A	D	$9.29*10^{-4}$	1925.985	-8.442E-05	5.6095E-05	-0.0004236
A	E	$1.16*10^{-3}$	4443.182	0.00013733	-0.0013717	-0.0006259
A	F	$9.94*10^{-4}$	2841.387	0.00018296	-0.000146	-0.0001337
B	C	$8.50*10^{-4}$	1746.001	0.00055742	-0.0002459	0.0007333
B	D	$9.29*10^{-4}$	2210.552	0.00011558	-4.391E-05	0.00057642
B	E	$1.16*10^{-3}$	4740.458	3.7334E-05	0.00062828	0.00037415
B	F	$9.94*10^{-4}$	3203.401	8.2956E-05	0.00085396	0.00086633

* For more information of calculation see appendix B

CHAPTER

7

Conclusions and Recommendations

This chapter covers the conclusions and recommendations of the project.

CHAPTER SEVEN

7.1 Introduction

After our project had been completed, we have the following conclusions and recommendations.

7.2 Conclusions

The conclusions are summarized as follows:

- The value of geoid separation was calculated using equation ($h = N + H$), N=15.906m.

- The value of (N) cannot be ignored in heighting, if (N>6m).

- The precise coordinate of Baseline was calculated using Least Squares technique then, the Grid distance was calculated depending on the precise coordinate, so that this distance will be considered as the most probable value.

- A precise Geodetic Network was established depending on the base line.

- The control points had been applied on map of Hebron city using (Geographic Information System) GIS software and the results were very encouraged.

- The least squares method that used in calculation was efficient.

- For any network, enough loop closures should be computed so that every baseline is including within at least one loop. This should expose any large blunder that exists. If a blunder does exist, its location can be often determined through additional loop closure analysis.

- We had good ideas about establishing geodetic networks using GPS techniques.

7.3 Recommendations

Our recommendations are summarized as follows:

- Compute deviation of the vertical (angle between normal to ellipsoid and normal to geoid)

- Densifcate geodetic control points to include all part of Hebron city.

- Establish general geodetic network for Hebron governorate include all towns and villages, then to establish it for other governorate in west bank.

- Extend the study surveying method using GPS.

- Using GPS in surveying in high way projects.

- To cooperate with the Palestine Geographic Center and other governmental establishing to make such control points.

A . 5700 GPS Receiver

A .1 Introduction

This chapter provides general setup information, connection information, and cabling diagrams for the most common uses of the 5700 receiver.

A . 2 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 A.1. This section provides a brief overview of the features of each of these panels.

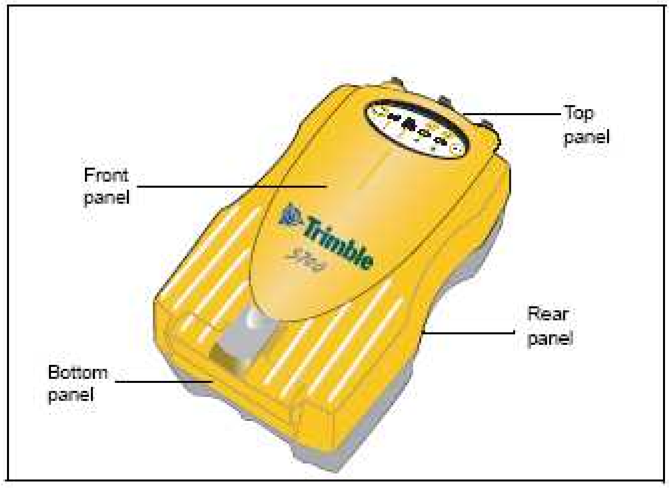


Figure A.1: Panels on the 5700 receiver

A . 3 Front panel

Figure A.2 shows the front panel of the 5700 receiver. This panel contains the five indicator LEDs, the two buttons, and the catch for the CompactFlash/USB door.

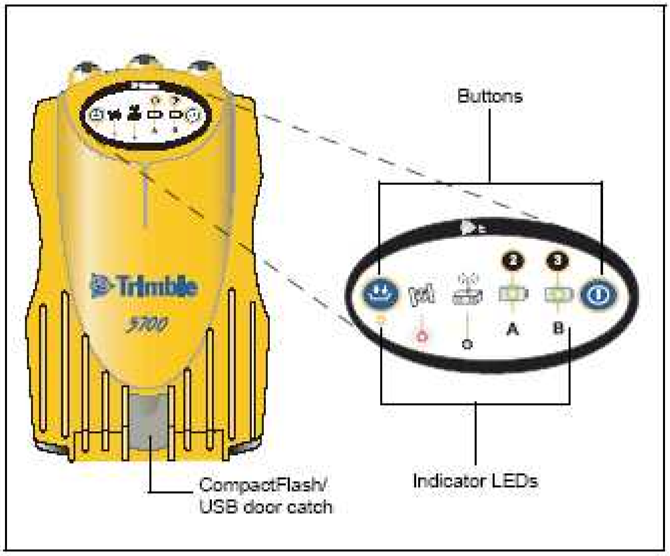


Figure A.2: Front panel.

A . 4 Top panel

Figure A .3 shows the top panel of the 5700 receiver. This panel contains the three power/serial data ports and (TNC) ports for GPS and radio antenna connections.

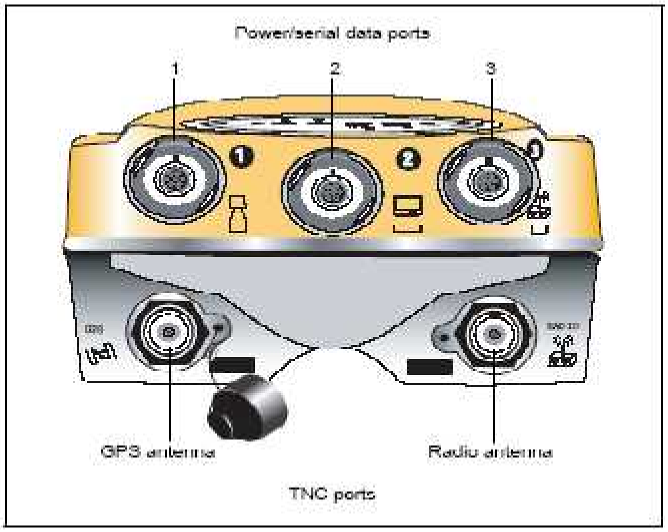
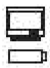
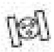


Figure A .3: Top panel

Each port on the top panel is marked with an icon to indicate its main function.

Table A .1 5700 receiver ports

Icon	Name	Connections
	Port 1	Trimble controller, event marker, or computer
	Port 2	Power in, computer, 1PPS, or event marker
	Port 3	External radio, power in
	GPS	GPS antenna
	RADIO	Radio communications antenna

.

A .4 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.

A .5 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

A . 6 Post processed Setup

For a post processed survey, you only need:

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

Other equipment, as described below, is optional.To set up the 5700 receiver for a post processed survey:

1. 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.

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

Figure A .4: Post processed setup

A . 7 Other System Components

This section describes optional components that you can use with the 5700 receiver.

A . 7 . 1 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.

A . 7 . 2 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 A .5 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.

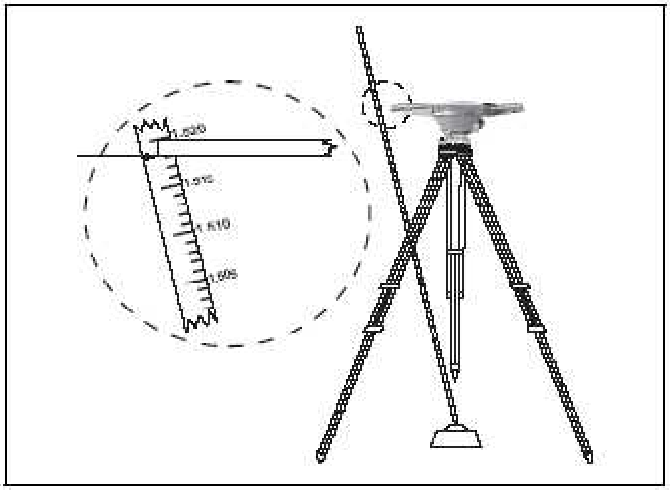


Figure A .5: Measuring antenna height

A . 8 Positioning Specifications (Measurements)

• Advanced Trimble Maxwell technology
• High-precision multiple correlator L1 pseudorange measurements
• Unfiltered, unsmoothed pseudorange measurement data for low noise, low multipath error, low time domain correlation, and high dynamic response
• Very low noise L1 measurements with <1 mm precision in a 1 Hz bandwidth
• L1 Signal-to-Noise ratios reported in dB-Hz
• Proven Trimble low-elevation tracking technology
• 12 Channels L1 C/A Code, L1 Full Cycle Carrier, WAAS/EGNOS1

Code differential GPS positioning

Horizontal . . . . ± (0.25 m + 1 ppm) RMS
Vertical . . . . ± (0.5 m + 1 ppm) RMS
WAAS differential positioning accuracy typically <5 m 3DRMS

Static and Fast Static GPS surveying

Horizontal . . . . ± (5 mm + 0.5 ppm) RMS

Vertical . . . .± (5 mm + 1 ppm (× baseline length)) RMS

Kinematic surveying

Horizontal . . . . ….. ± (10 mm + 1 ppm) (× baseline length) RMS

Vertical . . . . …. ± (20 mm + 1 ppm) RMS

RTK surveying

Horizontal . . . . ……... ± (1 cm + 1 ppm (× baseline length) RMS

Vertical . . . . ….. ± (2 cm + 1 ppm (× baseline length) RMS

B. Trimble Geomatics Office

B.1 Introduction

Geomatics is the design, collection, storage, analysis, display, and retrieval of spatial information. The collection of spatial information can be from a variety of sources, including GPS and terrestrial methods. Geomatics integrates traditional surveying with new technology-driven approaches, making geomatics useful for a vast number of applications.

Trimble Geomatics Office is a link and survey reduction package. It provides a seamless link between your field work and design software. The software includes an extensive feature set which helps you to verify your field work quickly, and easily perform survey-related tasks and export your data to a third-party design package.

B . 2 The Trimble Geomatics Office Graphics Window

When you start Trimble Geomatics Office, the main graphics window opens in the Survey view. The Trimble Geomatics Office graphics window contains standard Microsoft Windows functionality such as menus, shortcut menus, and toolbars, as well as a number of special features. Some of these items change according to which view you are using to display data in the graphics window: the default Survey view or the Plan view.

To become familiar with all these items, use the software's ToolTips, or access the Trimble Geomatics Office Help by pressing [F1].Figure A .6 shows the graphics window, including the features that are common to both views of the software,.

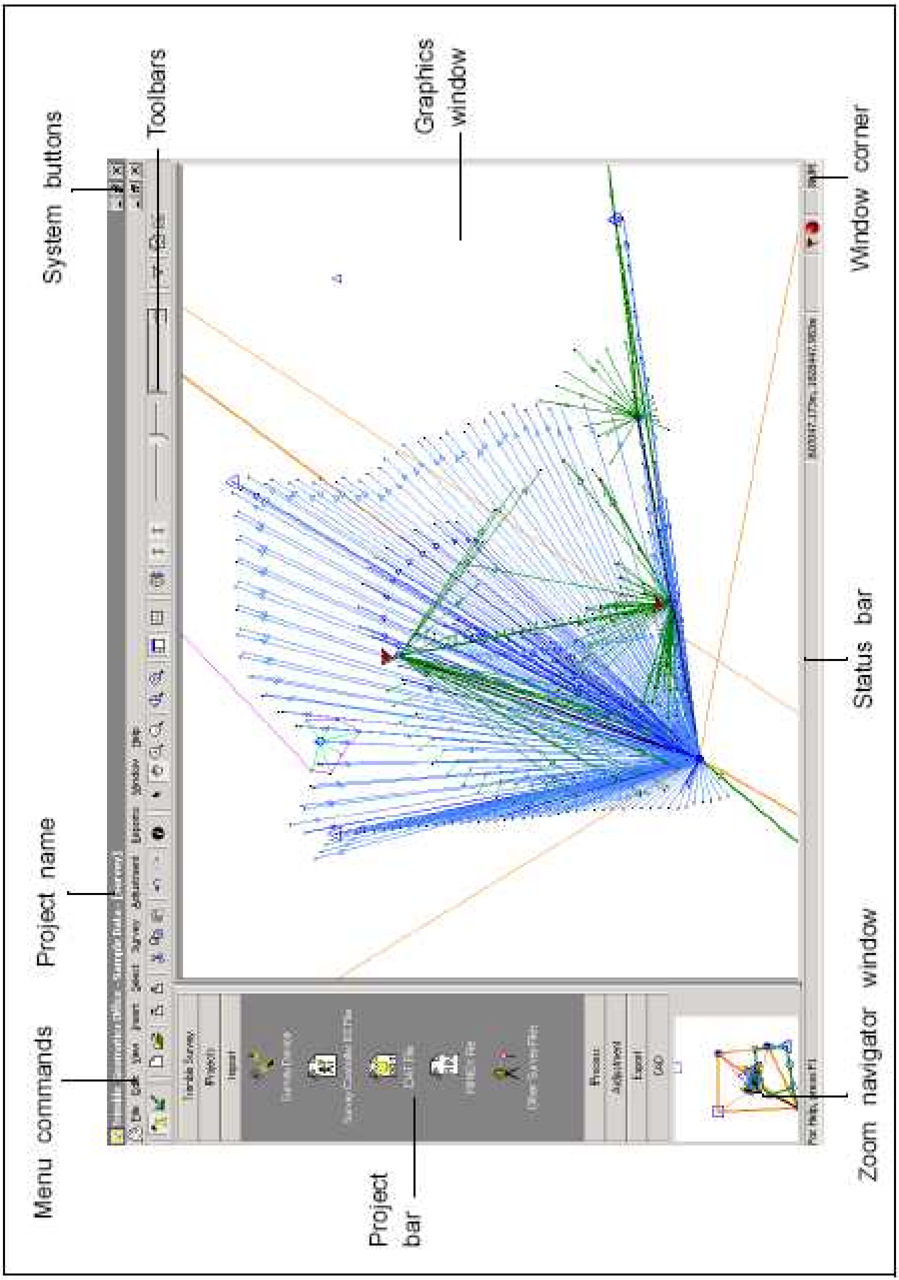


Figure A .6: Parts of the graphics window common to both views

B.3 Network Adjustment

B.3.1 Introduction

When surveying, you should collect extra data so that you can check the integrity of your observations. When a survey has extra observations (redundancy), you can use them to minimize the effects of inherent errors before producing final results.

The Network Adjustment module helps you to do the following:

- Detect blunders and large errors in your measurements
- Account for systematic errors
- Estimate and model random errors
- Constrain your measurements to a published or assumed coordinate system, so that you can account for datum transformations.

B.3.2 Network Adjustment Workflow

Two major steps are used to perform a network adjustment:

- The minimally constrained adjustment
- The fully constrained adjustment

This chapter describes the procedures for both steps, starting with a minimally constrained adjustment, then moving on to the fully constrained adjustment. Figure A .7 shows the typical workflow for a minimally constrained adjustment, and the following sections give more information about each step.

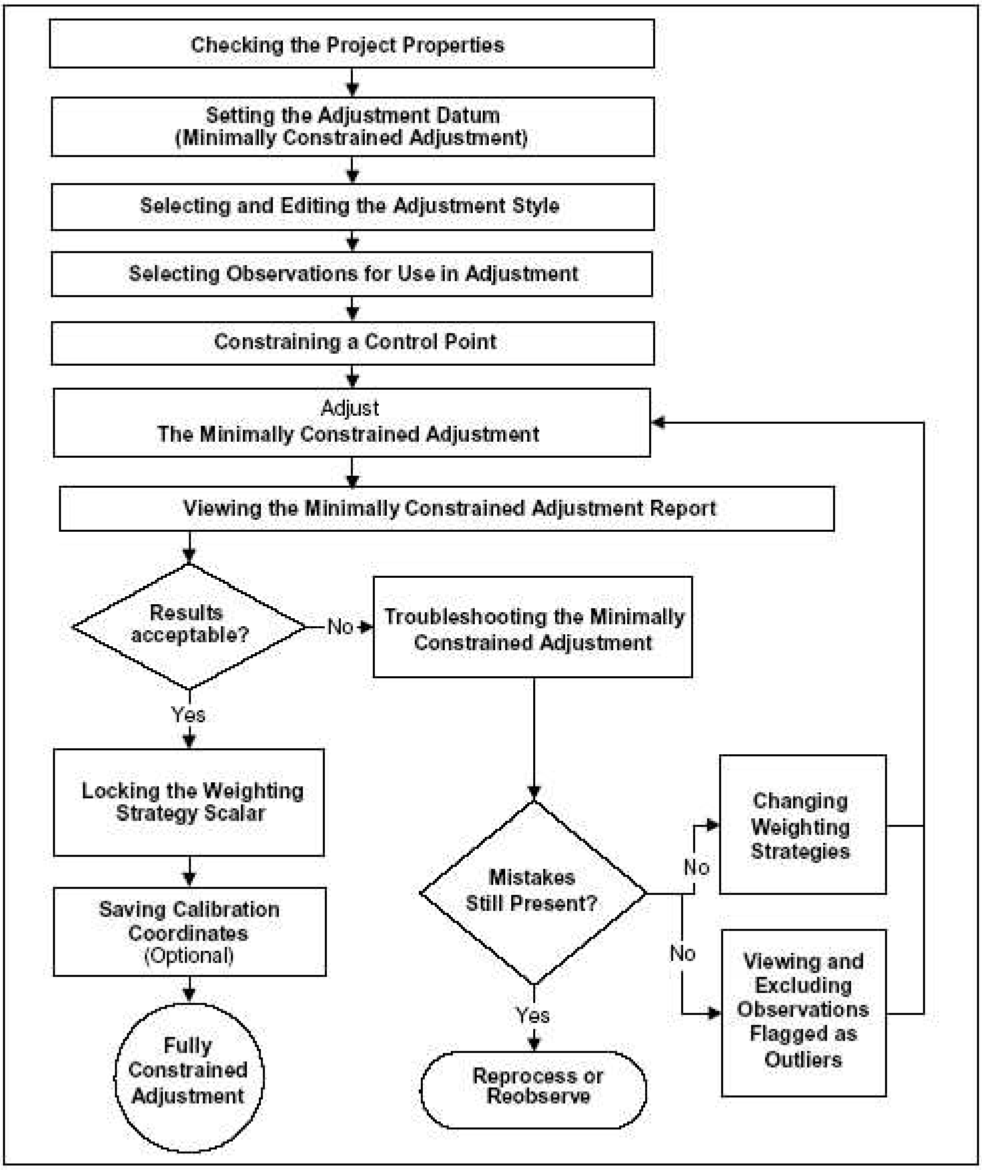


Figure A .7: Minimally constrained adjustment flow

B.4 Figures from Trimble Geomatics Office Software

After completed geodetic network we obtained many figures from Trimble geomatics office software these which described as follows the motion of satellite, the time taken the observation, and PDOP.

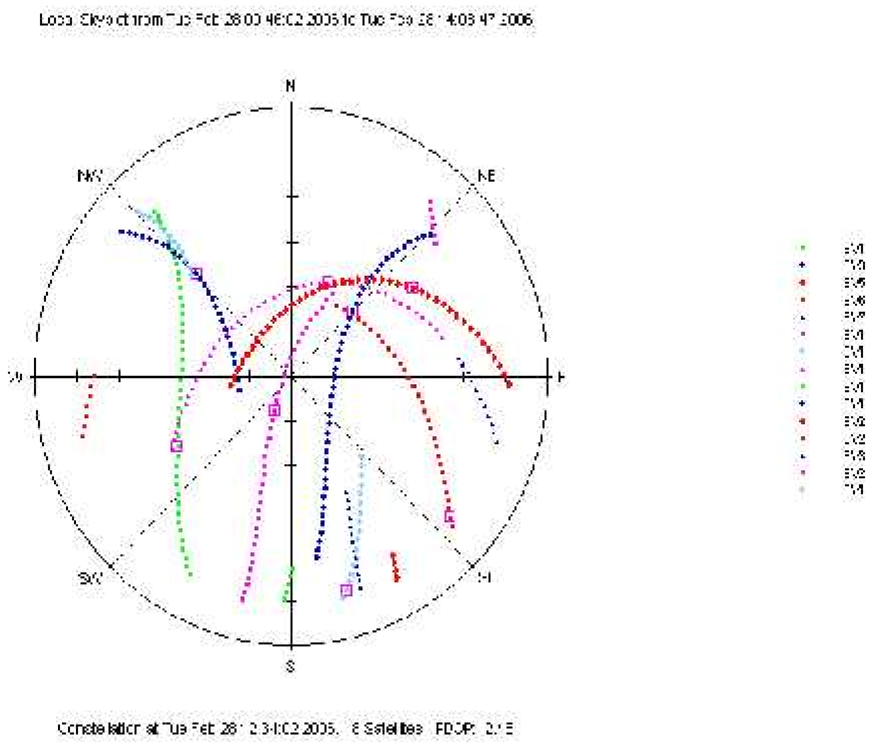


Figure A .8: Occupation Skyplot for point A

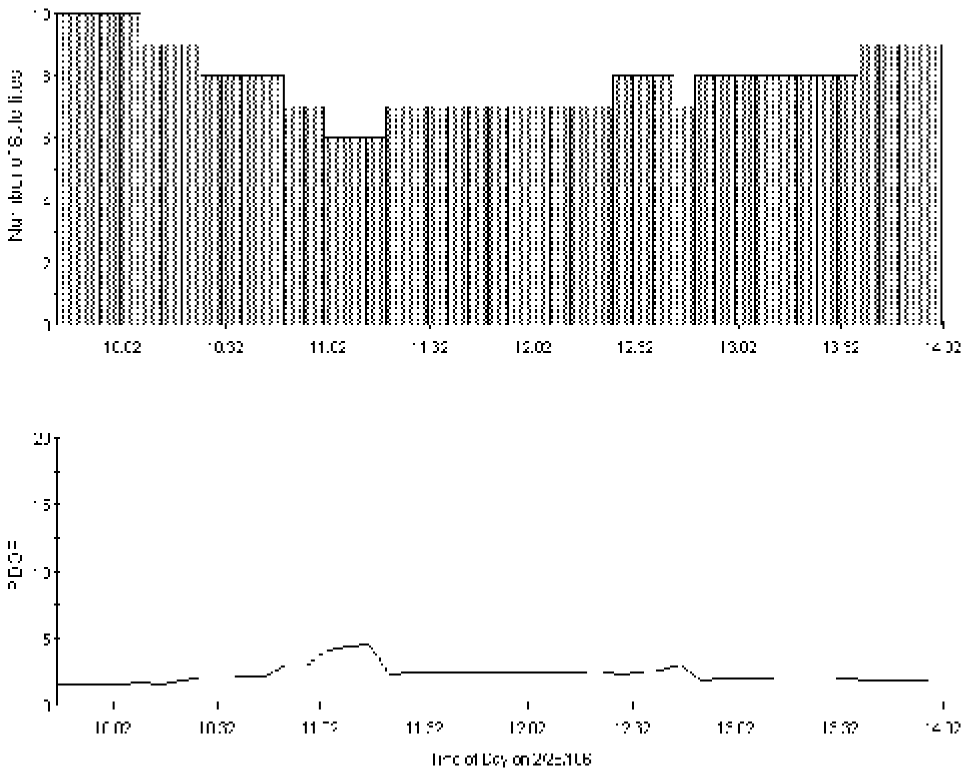


Figure A .9: Occupation DOP/ SV Plot for point A

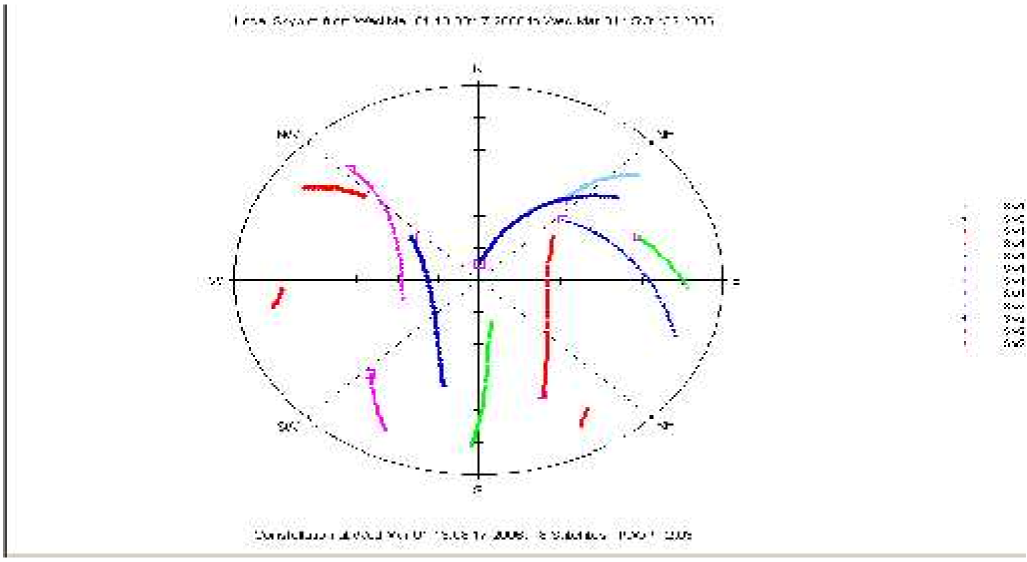


Figure A .10: Occupation Skyplot for point B

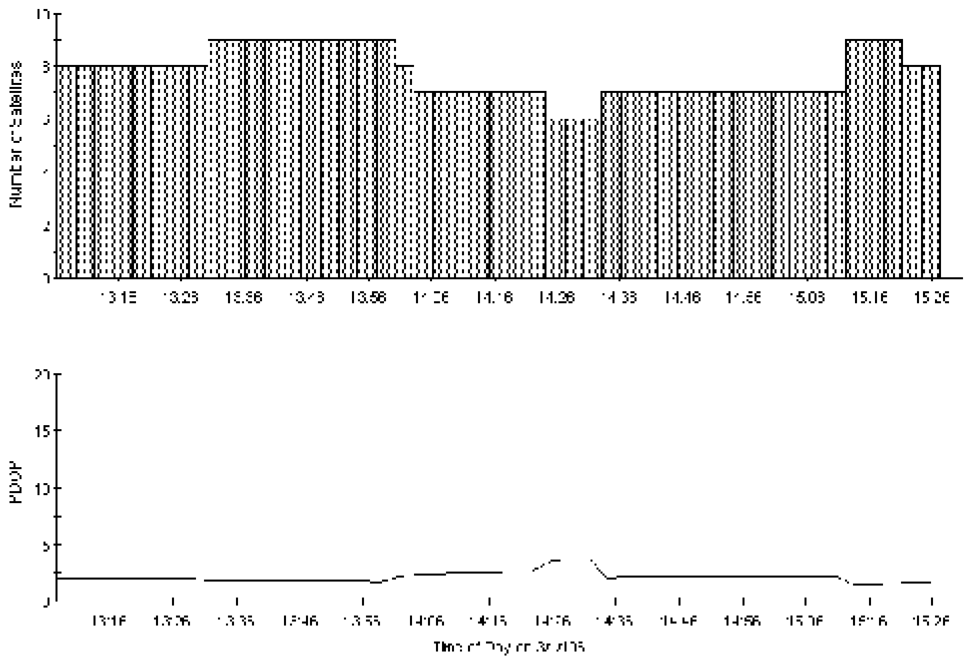


Figure A .11: Occupation DOP/ SV Plot for point B

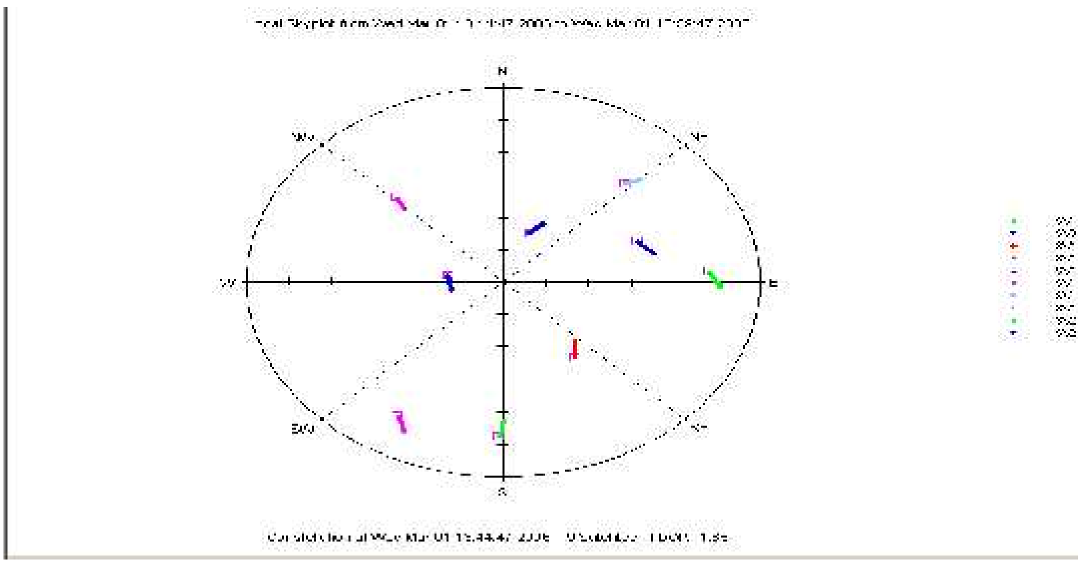


Figure A .12: Occupation Skyplot for point C

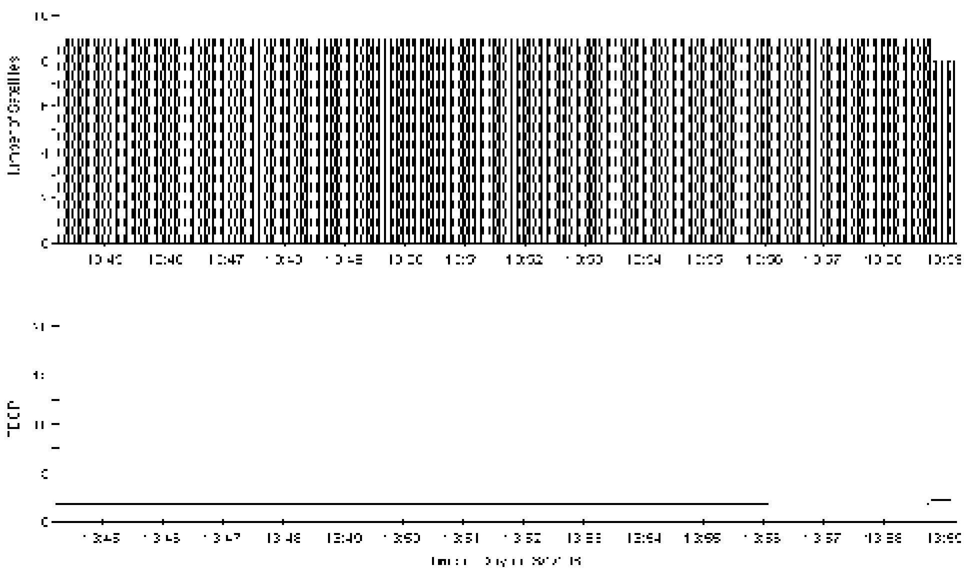


Figure A .13: Occupation DOP/ SV Plot for point C

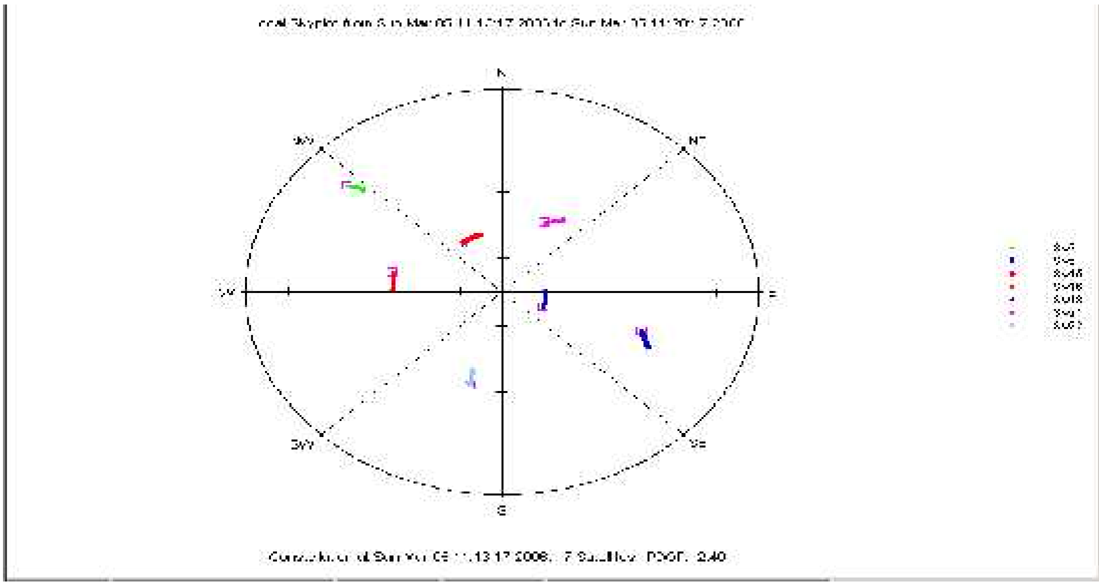


Figure A .14: Occupation Skyplot for point D

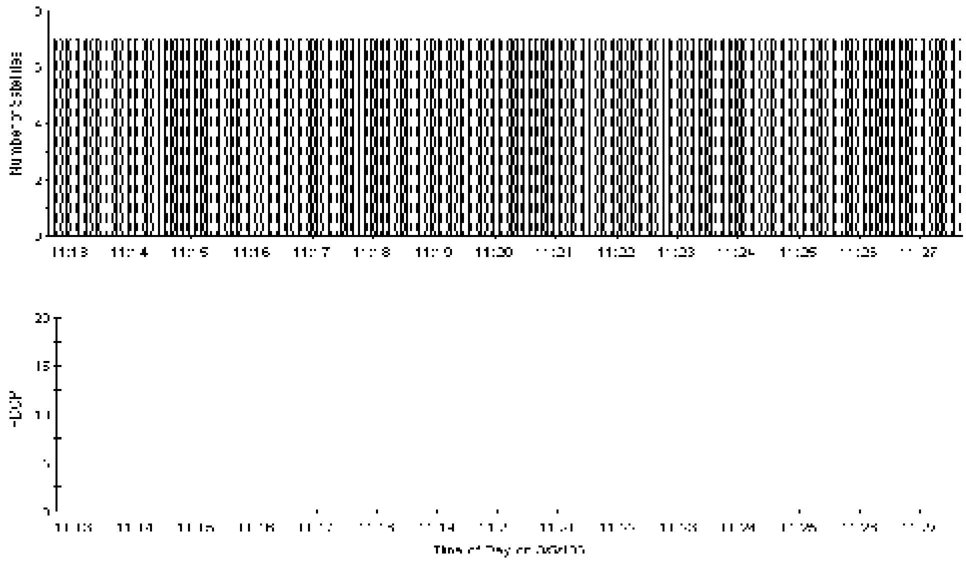


Figure A .15: Occupation DOP/ SV Plot for point D

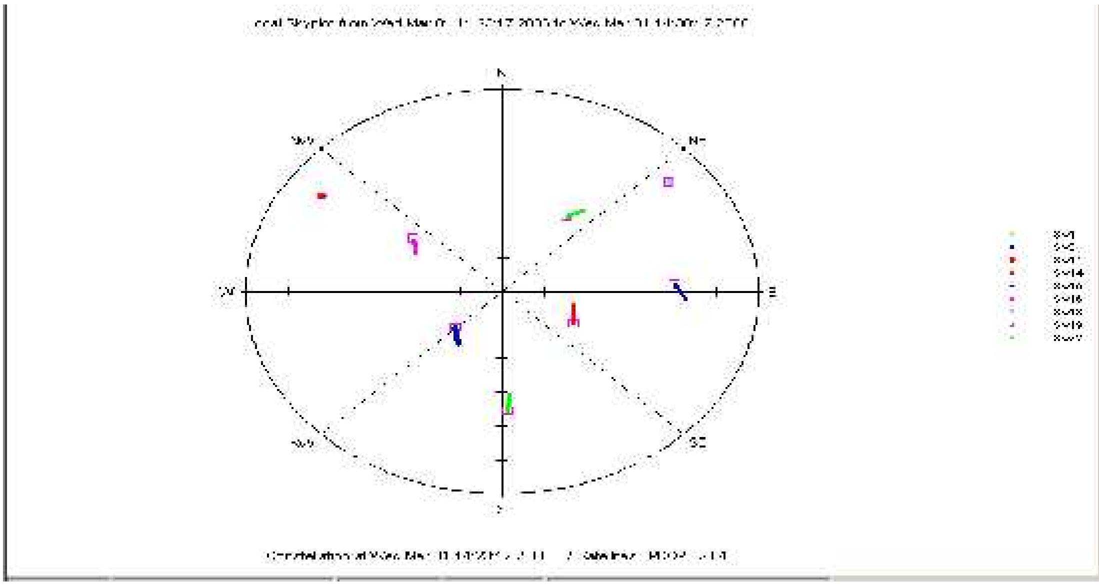


Figure A .16: Occupation Skyplot for point E

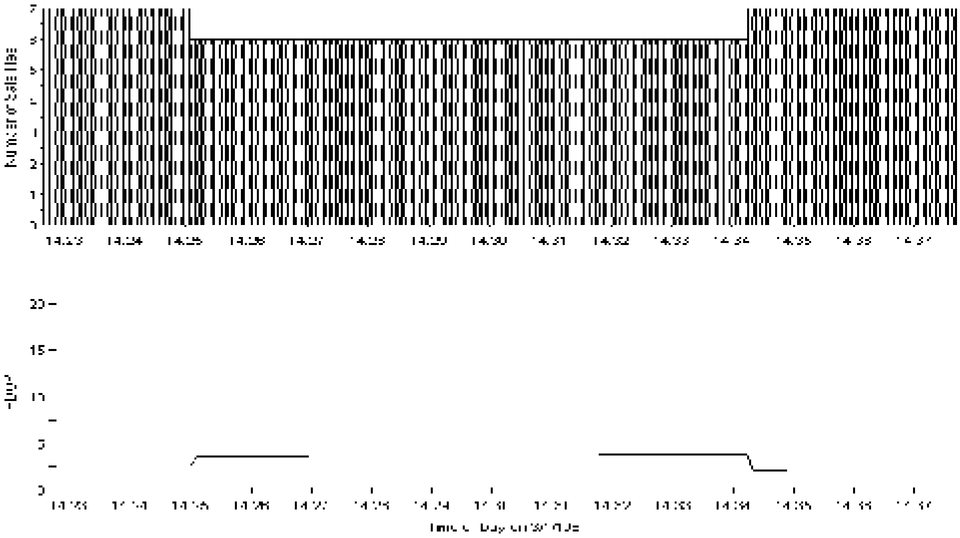


Figure A .17: Occupation DOP/ SV Plot for point E

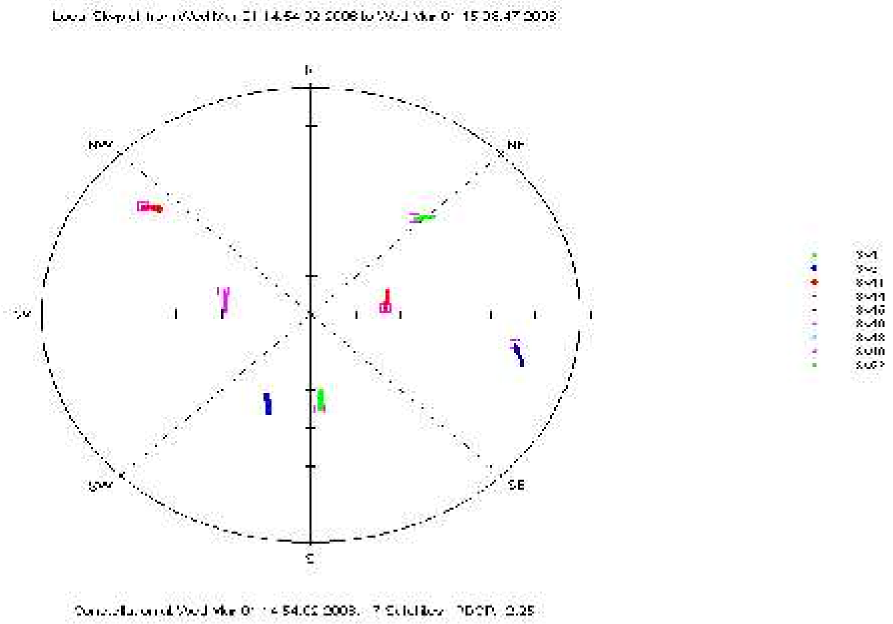


Figure A .18: Occupation Skyplot for point F

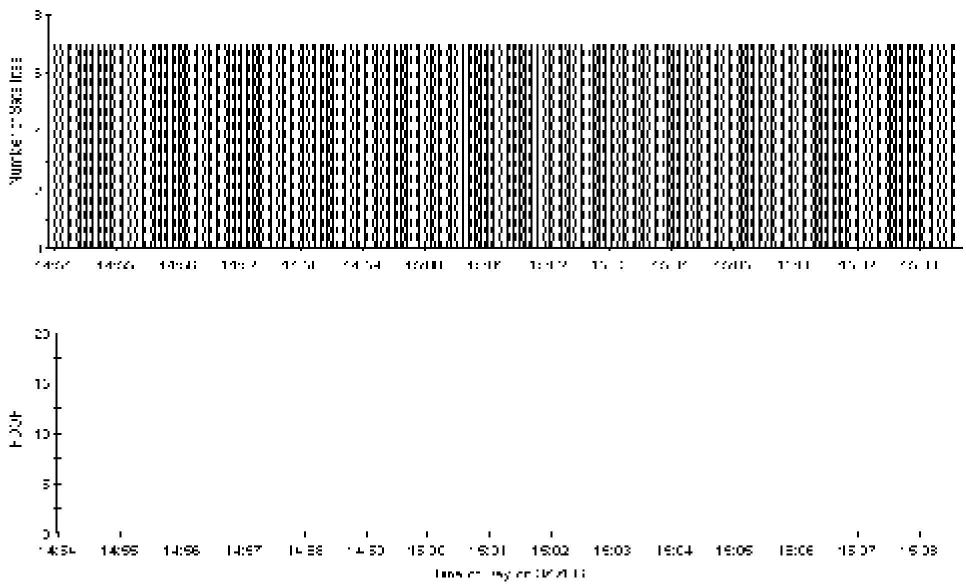


Figure A .19: Occupation DOP/ SV Plot for point F

Table 6.3 Difference between measured and fixed coordinates

From "1"	**To "2"**	ΔX **(m) "3"**	ΔY **(m) "4"**	ΔZ **(m) "5"**
A	C	-428.464	1415.792	50.832
A	D	-388.555	1885.954	40.283
A	E	-514.631	4413.151	33.430
A	F	512.480	2794.787	-3.301
B	C	-189.3060	1734.9751	50.419
B	D	-149.3962	2205.1371	39.871
B	E	-275.4719	4732.332	33.017
B	F	751.6391	3113.9691	-3.713

Table 6.4 Covariance matrix elements

Name Point	**CovarianceXX** $(\pm_{X^2})$	**CovarianceXY** $(\pm_{XY})$	**CovarianceXZ** $(\pm_{XZ})$	**CovarianceYY** $(\pm_{Y^2})$	**CovarianceYZ** $(\pm_{YZ})$	**CovarianceZZ** $(\pm_{Z^2})$
A – C	2.3554013713592E-06	1.18176387043734E-06	8.68379456875357E-07	9.58675576379915E-07	6.25168678623691E-07	7.33270174196438E-07
A – D	2.04379794371781E-06	1.10880856670842E-06	1.33942997555606E-06	1.22865240851327E-06	7.9541663912843E-07	1.51398284112124E-06
A – E	5.02689445144031E-06	3.35649939300284E-06	2.04388088405271E-06	5.01837075302884E-06	1.96935324735371E-06	3.10310896813964E-06
A – F	2.18657816976199E-06	1.69102229867256E-06	1.00194469219735E-06	2.16166346699171E-06	7.06420618619475E-07	1.08962801871588E-06
B – C	3.50248990307111E-06	2.31337935948609E-06	1.61292737138513E-06	3.61908575545262E-06	1.75224304090988E-06	2.4627422149242E-06
B – D	5.16092532875395E-06	2.25280276905812E-06	3.01135032111595E-06	1.93206367149969E-06	1.51884348131087E-06	2.67780589960167E-06
B – E	3.12967262450989E-06	1.95219427627656E-06	2.39615885982614E-06	2.36457437262516E-06	1.75750569772091E-06	3.13898697662298E-06
B - F	4.91360102606749E-06	3.71683827132785E-06	3.61508990015467E-06	4.37553921724257E-06	3.29703778575669E-06	4.21799343511029E-06

WEIGHT MATRIX

By definition W = $\sum^{-1}$. In this problem covariance matrix is:

$$\sum = \begin{bmatrix}
\pm^{2}_{X_{AC}} & \pm_{XY_{AC}} & \pm_{XZ_{AC}} & 0 & 0 & 0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 \\
\pm_{XY_{AC}} & \pm^{2}_{Y_{AC}} & \pm_{YZ_{AC}} & 0 & 0 & 0 & 0 & 0 & 0 & & 0 & 0 & 0 \\
\pm_{XZ_{AC}} & \pm_{YZ_{AC}} & \pm^{2}_{Z_{AC}} & 0 & 0 & 0 & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & \pm^{2}_{X_{AD}} & \pm_{XY_{AD}} & \pm_{XZ_{AD}} & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & \pm_{XY_{AD}} & \pm^{2}_{Y_{AD}} & \pm_{YZ_{AD}} & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & \pm_{XZ_{AD}} & \pm_{YZ_{AD}} & \pm^{2}_{Z_{AD}} & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & \pm^{2}_{X_{AE}} & \pm_{XY_{AE}} & \pm_{XZ_{AE}} & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & \pm_{XY_{AE}} & \pm^{2}_{Y_{AE}} & \pm_{YZ_{AE}} & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & \pm_{XZ_{AE}} & \pm_{YZ_{AE}} & \pm^{2}_{Z_{AE}} & & 0 & 0 & 0 \\
\vdots & & & & & & & & & \ddots & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \pm^{2}_{X_{BF}} & \pm_{XY_{BF}} & \pm_{XZ_{BF}} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \pm_{XY_{BF}} & \pm^{2}_{Y_{BF}} & \pm_{YZ_{BF}} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \pm_{XZ_{BF}} & \pm_{YZ_{BF}} & \pm^{2}_{Z_{BF}}
\end{bmatrix}$$

$$
\Sigma = \begin{bmatrix}
2.3554*10^{-6} & 1.1817*10^{-6} & 8.6837*10^{-7} & 0 & 0 & 0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 \\
1.1817*10^{-6} & 9.5867*10^{-7} & 6.2516*10^{-7} & 0 & 0 & 0 & 0 & 0 & 0 & & 0 & 0 & 0 \\
8.6837*10^{-7} & 6.2516*10^{-7} & 7.3327*10^{-7} & 0 & 0 & 0 & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & 2.0437*10^{-6} & 1.1088*10^{-6} & 1.3394*10^{-6} & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & 1.1088*10^{-6} & 1.2286*10^{-6} & 7.9541*10^{-7} & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & 1.3394*10^{-6} & 7.9541*10^{-7} & 1.5139*10^{-6} & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 5.0268*10^{-6} & 3.3564*10^{-6} & 2.0438*10^{-6} & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 3.3564*10^{-6} & 5.0183*10^{-6} & 1.9693*10^{-6} & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 2.0438*10^{-6} & 1.9693*10^{-6} & 3.1031*10^{-6} & & 0 & 0 & 0 \\
\vdots & & & & & & & & & \ddots & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 4.9136*10^{-6} & 3.7168*10^{-6} & 3.6151*10^{-6} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 3.7168*10^{-6} & 4.3755*10^{-6} & 3.297*10^{-6} \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 3.6151*10^{-6} & 3.297*10^{-6} & 4.2179*10^{-6}
\end{bmatrix}
$$

$$
W = \begin{bmatrix}
1148436.528 & -1189626 & -345183.34 & 0 & 0 & 0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 \\
-1189626.489 & 3581964 & -1646001.2 & 0 & 0 & 0 & 0 & 0 & 0 & & 0 & 0 & 0 \\
-345183.3406 & -1646001 & 3176076.19 & 0 & 0 & 0 & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & 1523511.822 & -762035.6458 & -947692.7808 & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & -762035.6458 & 1615954.52 & -175125.2451 & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & -947692.7808 & -175125.2451 & 1591708.694 & 0 & 0 & 0 & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 391274.5653 & -213943.6268 & -121659.8892 & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & -213943.6268 & 382192.2011 & -101496.0149 & & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & -121659.8892 & -101496.0149 & 466022.1387 & & 0 & 0 & 0 \\
\vdots & & & & & & & & & \ddots & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 728370.1 & -360750.6037 & -342343.6434 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -360750.6 & 734758.5448 & -265208.7953 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -342343.6 & -265208.7953 & 737957.2372
\end{bmatrix}
$$

References

1. James M.Anderson & Edowarad M.Mikhail, Surveying Theory and Practice, Fifth Edition, 1998.
2. Leachier Notes in Cartography, By Eng. Faydi SHabaneh, Second Semester 2004-2005,PPU
3. Paul R. Wolf, Adjustment Computations Statistics and Least Squares in Surveying and GIS, John Wiley & Sons, Inc., Canada, 1997.
4. W.Schofield, Engineering Surveying Theory and Examination Problems for Students, Principal Lecturer, Kingston University.
5. Trimble Geomatics Office User Guide
6. 5700/5800 GPS Receiver User Guide
7. http://activetectonics.la.asu.edu/TotalStation/document.html
8. http://www.cmtinc.com/gpsbook/index.htm
9. http://www.gmat.unsw.edu.au/snap/gps/gps_survey/principles_gps.htm
10. http://www.rbf.com/cgcc/
11. http://www.sokkia.net
12. http:// www.surveyingsupplies.com/surveyingsupplies/pdfs/gpsbasics.pdf
13. http://www.terrasurv.com/edm.
14. http://www.trimble.com/

Appendix A

The following contents are going to be covered in this Appendix:

A . 5700 GPS Receiver

B . Trimble Geomatics Office

Appendix

The following contents are going to be covered in this Appendix:

Appendix A

A . 5700 GPS Receiver

B . Trimble Geomatics Office

Appendix B

All Calculation in Software Microsoft Excel by using Least Squares